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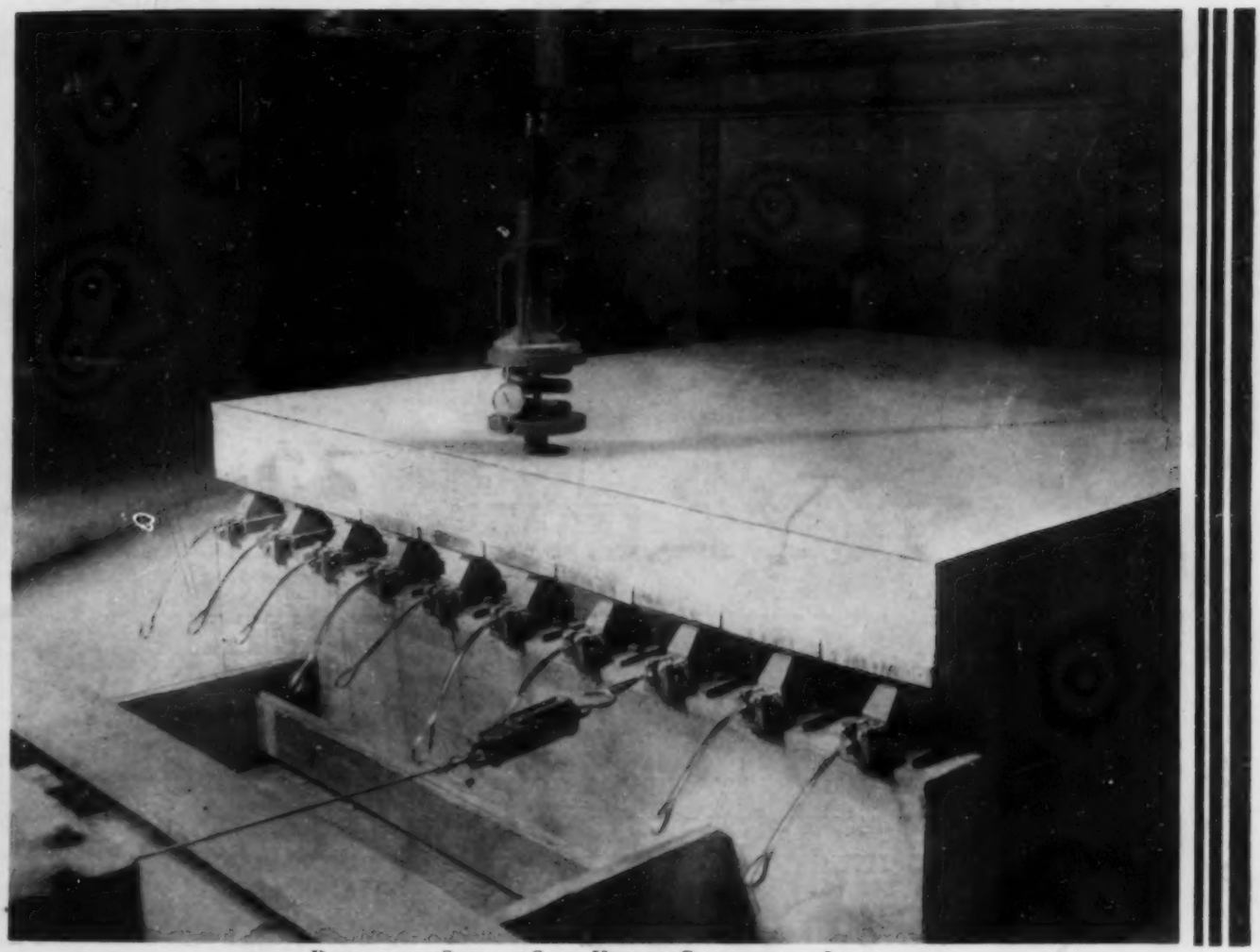

DETERMINING SHEAR IN SLABS UNDER A CONCENTRATED LOAD
Reactions Measured by Friction Tapes

Volume 4 — Number 11

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

NOVEMBER 1934

AMERICA GOES DIESEL · AMERICA GOES DIESEL · AMERICA GOES DIESEL · AMERICA GOES DIESEL ·

▲ This bulldozer-equipped "Caterpillar" Diesel Seventy-Five Tractor supplies low-cost, dependable power to Contractor Clarence Young for a mountain road building job near Butte Meadows, California.

▼ This is one of three "Caterpillar" Diesel Seventy-Five Tractors on Mr. R. Roland's contract at Coulee Dam. Working 20 hours per day. Moving 11 to 12 yards per trip. Fuel cost is only about 30 cents per hour.

◀ Pulling a "Caterpillar" Elevating Grader, this "Caterpillar" Diesel Fifty Tractor is doing canal construction work for the Irrigation Department of the Union of South Africa.

"SO WE INCREASED OUR FLEET OF 'CATERPILLAR' DIESEL TRACTORS"

—SAYS H. W. McKINLEY, GLENDALE, CALIFORNIA, CONTRACTOR, AFTER HIS COST RECORDS SHOWED THAT THREE BIG "CATERPILLAR" DIESEL TRACTORS WERE COSTING LITTLE MORE FOR FUEL THAN ONE 30-HORSEPOWER GAS TRACTOR.

Another southern California contractor, Clarence Crow, of Crow Brothers, writes: "Our fuel cost for each of our 'Caterpillar' Diesel Seventy-Five Tractors is only one-fifth of what it used to be for our big gas tractors."

Other owners, everywhere, report similar results. Says E. B. Boyle, President of the Boyle Road and Bridge Company of Sumter, S. C.: "Fuel for the 'Caterpillar' Diesel costs us less than 1/3 the cost of fuel for a gas machine of the same size." From Pickering, Mo., Mr. L. O. Killion writes: "Ten to twelve gallons of fuel is driving the Diesel Thirty-Five the same number of hours that 25 gallons of gasoline did the gas 30-h. p. tractor, besides doing one-half more work."

More work—less cost—such are the results reported by the thousands of enthusiastic owners of "Caterpillar" Diesel Tractors and Power Units. That explains why more and more users of tractors and stationary power are going Diesel—with "Caterpillar." Caterpillar Tractor Co., Peoria, Illinois, U. S. A.

Among Our Writers

HUNTER ROUSE was sent to Europe by the Massachusetts Institute of Technology as a Traveling Fellow in Hydraulics and spent two years, 1929–1931, studying principally at Kalsruhe, under Th. Rehbock and at the outdoor station near Walchensee, Bavaria. He described the work being done at the Walchensee laboratory in the May 1931 issue of CIVIL ENGINEERING. He then spent two years in the River Hydraulics Laboratory at M.I.T. He is now at Columbia University developing a laboratory for hydraulic research in fluid mechanics under Prof. B. A. Bakhmeteff.

RALPH W. POWELL graduated from Michigan State College in 1911. After further study at Cornell and Yale, he went to the college of Yale in China, where he taught applied mathematics from 1916 to 1927. Since 1927 he has been a member of the Department of Mechanics at Ohio State University. He is now on a year's leave of absence handling hydraulic and hydrology problems for the Muskingam Watershed Conservancy District, at New Philadelphia, Ohio.

DON HULL MCCREERY, after graduation from Massachusetts Institute of Technology in 1922, went with the Union Oil Company of California for six months on the design of 750,000-gal oil storage reservoirs. Since then he has been with the Richards-Neustadt Construction Company as engineer, in charge of construction of commercial buildings, hospitals, bridges, and churches. Recently completed projects of importance are the $800,000 First Congregational Church in Los Angeles and modernization of the J. W. Robinson Company department store.

LYNN PERRY will be remembered by his former colleagues on the New York Board of Water Supply and the New Jersey State Department of Health. After serving in France during the War, he went to Lafayette College, Easton, Pa., in charge of sanitary engineering subjects. He spends much of his time at his home on the "Eastern Shore" and is an authority on the local history of that early colonial settlement.

WILLARD E. SIMPSON has maintained a consulting office in San Antonio since 1919, confining his practice to the design of difficult foundations and structural frames of buildings and bridges. His most important structures have been the Smith-Young Tower in San Antonio, the Gulf Building in Houston, the reinforced concrete bridge across the Rio Grande at Laredo, and buildings at Randolph Field, an $11,000,000 project near San Antonio. He is now preparing structural plans for post offices at San Antonio, Galveston, and Austin, Tex., whose aggregate cost is $2,700,000.

WALTER A. WEISKOPF, a graduate of Rensselaer Polytechnic Institute, got his first structural steel experience with the American Bridge Company and the Carnegie Steel Company, by whom he was employed for eight years. With his present firm since 1926, he has engaged in the design of structural frames and foundations for buildings, notably the Mitsui Bank Building in Tokyo and the Federal Building in Pittsburgh.

JOHN W. PICKWORTH was graduated from Cooper Union in 1916. In 1919, after service in the Corps of Engineers, A.E.F., he formed the firm of Weiskopf and Pickworth, which has designed numerous buildings in New York City, among them being the Stock Exchange, the Equitable Trust, the Bank of America, and the Daily News buildings, as well as other important structures elsewhere.

M. G. SPANGLER, a graduate of Iowa State College in 1919, was a bridge designer for the Iowa State Highway Commission prior to 1924, when he became associated with the Iowa Engineering Experiment Station, for which he is now Associate Structural Engineer.

WILBUR R. BARROWS served 4 years with the Middle Rio Grande Conservancy District of Albuquerque, N.M., as transitman, office engineer, and senior draftsman. He is now employed as hydraulic designer by the Harza Engineering Company, at Columbus, Nebr.

A. W. NEWCOMER, after graduation from the University of New Mexico in 1926, joined the engineering staff of the Middle Rio Grande Conservancy District in New Mexico working on drainage, flood-control, and irrigation problems, among them the multiple-weir drop described in this issue. He is now in the Denver office of the U. S. Bureau of Reclamation.

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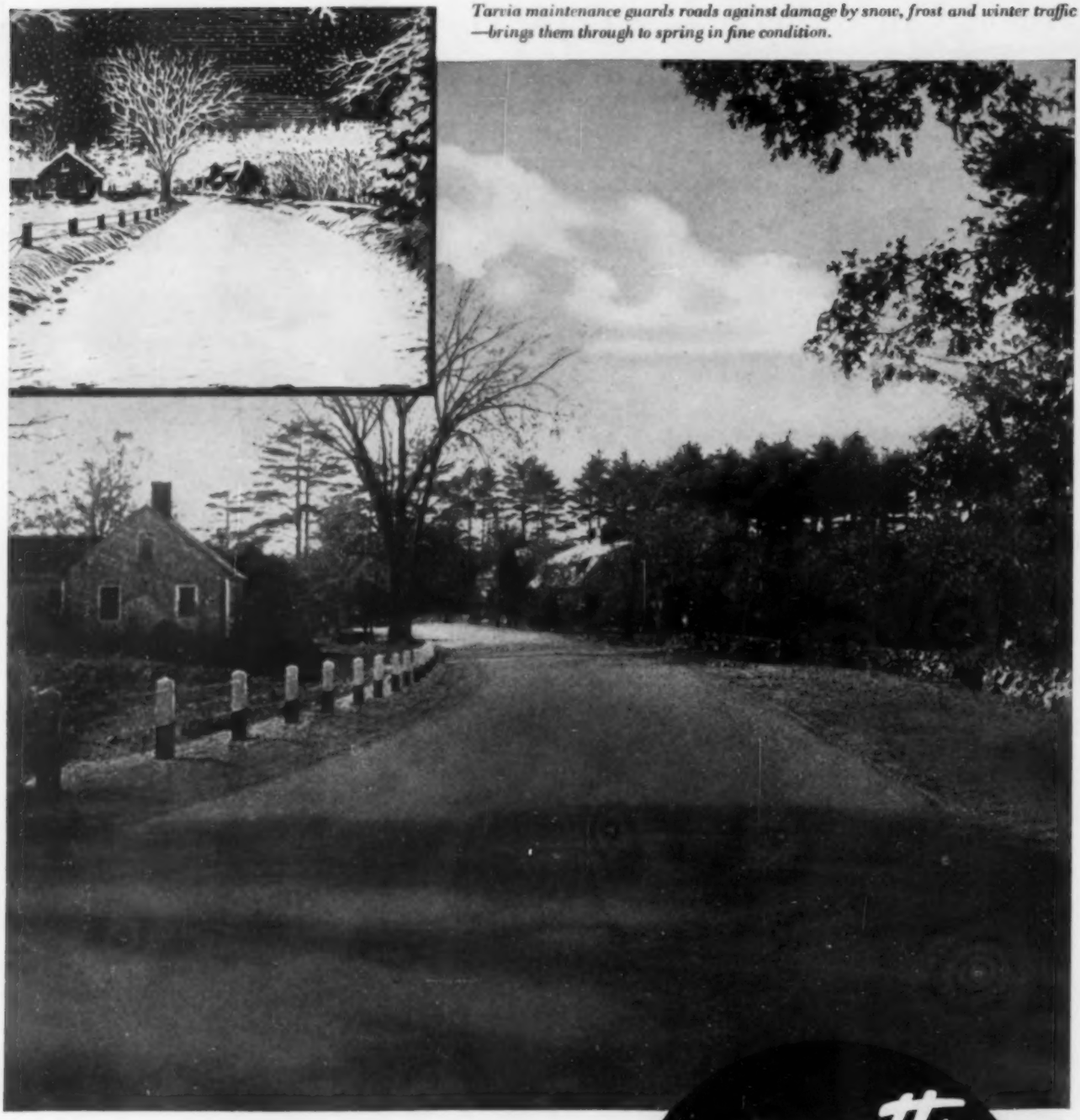

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VOLUME 4 NUMBER 11

The Analytical Approach to Experimental Hydraulics

A Brief Résumé of the Mathematical Relationships Between Similar Systems of Flow

IT was nearly 250 years ago that the great natural philosopher, Sir Isaac Newton, expressed in his "Principia" the general law of similarity, on which all model testing of today is based. About 175 years later, in 1872, Newton's concise and basic idea was applied by Froude in his studies of friction on ships' hulls. Froude's name has been perpetuated in current works on model analysis by his law expressing the relation of velocities for gravitational similarity by a dimensionless number **F**, *called the Froude number. In 1883 another English experimenter, Osborne Reynolds, enunciated a law expressing the relation between fluid density, viscosity, and velocity, and a length parameter. This relation, the Reynolds number, has been designated* **R**. *A similarity law involving the bulk modulus of elasticity of fluids is attributed to Cauchy and is expressed by a dimensionless number,* **C**, *known as the Cauchy number. Whenever dimensions become small, surface tension and capillary forces must be considered. The law of capillary similarity, attributed to Weber, involves a dimensionless ratio,* **W**, *or the Weber number.*

Although in recent years much work has been done with models and the results of many tests have been published in technical literature, it is appropriate to reenunciate and emphasize some of the more important principles of dynamic similarity and the use of the dimensionless numbers in model analysis to encourage still further use of this valuable tool. The author of the first article in this symposium, Dr. Rouse, received a Massachusetts Institute of Technology Traveling Fellowship in Hydraulics in 1929 and spent the subsequent two years studying in European laboratories. The second, by Professor Powell, was originally delivered at the Models Conference of the Society for the Promotion of Engineering Education at Cornell University in June 1934.

On the Use of Dimensionless Numbers

By HUNTER ROUSE
INSTRUCTOR IN CIVIL ENGINEERING, COLUMBIA UNIVERSITY, NEW YORK, N.Y.

WHILE the investigations of Osborne Reynolds presented in his paper, "The Two Manners of Motion of Water," read March 28, 1884, before the Royal Institution of Great Britain (*Proceedings* of that institution, Vol. 11), are commonly considered to have begun a new era for the science of fluid mechanics, the majority of hydraulicians were slow to avail themselves of the very practical tool he had devised. Although similar trends of thought were followed by such physicists and engineers of the last century as Bertrand, Froude, Reech, Helmholtz, and Rayleigh, the practicability of their methods was still so obscure that hydraulicians continued for nearly fifty years to rely on empirical formulas, many of which are noteworthy for their lack of dimensional homogeneity.

LETTERS AND SYMBOLS USED IN THIS ARTICLE

d = increment of
g = acceleration of gravity
h = head
m = mass
t = time
C = weir coefficient
F = Froude number
F = force
I = inertial reaction to any applied force = $\frac{d(mV)}{dt}$
L = Length parameter
Q = rate of discharge
R = Reynolds number
V = velocity
W = Weber number
γ (gamma) = specific weight of any fluid
μ (mu) = viscosity of a fluid
ν (nu) = kinematic viscosity of a fluid = $\frac{\mu}{\rho}$
ρ (rho) = fluid density = $\frac{\gamma}{g}$
σ (sigma) = surface tension of a fluid

Not until the last decade, after the Kármán-Prandtl school of aerodynamics had applied its theories of turbulence to pipe flow, did hydraulicians in general begin to look with great favor upon the study of fluid motion through the laws of dynamic similarity and the use of dimensionless numbers. One of the first American engineers to advocate the use of similitude in pipe design was W. F. Durand, in "The

Hydraulics of Oil Pipe Lines," published in the *Journal of Electricity*, Vol. 44 (1920), No. 9. (An admirable résumé of the historical growth of the science is given in Durand's "The Development of Our Knowledge of the Laws of Fluid Mechanics," in *Science* for October 20, 1933.) By the time *Hydraulic Laboratory Practice* was being prepared for the press by the late John R. Freeman, Past-President and Hon. M. Am. Soc. C.E., in 1928–1929, interest in the subject had increased to such an extent that several chapters were added to the original German edition, dealing in detail with dynamic similarity and dimensional analysis. This was soon followed by a widespread discussion of the comprehensive paper by B. F. Groat, M. Am. Soc. C.E., on "Theory of Similarity and Models," published in TRANSACTIONS, Vol. 96 (1932).

With the advent of research by means of hydraulic models in numerous technical schools throughout the world, traditional methods of instruction were gradually augmented by the inclusion of the elements of similitude, thus further popularizing this comparatively modern approach to the study of hydraulic phenomena. Yet so eager is the young craftsman to seize upon a new tool, once its worth has been proved, that he often overestimates its strength in his enthusiasm. Not only are most hydraulicians of the present generation now distrustful of dimensionally incorrect expressions, but many have acquired implicit faith in all dimensionless quantities, regardless of the possibility of incomplete physical justification.

In order to clarify the following discussion, a brief review of the development of three general relationships essential to dynamic similarity will be given. There are three fundamental forces that may influence the motion of incompressible fluids: gravity, viscosity, and molecular attraction (surface tension). Each of these, either independently or in combination with one or both of the others, may act to accomplish a rate of change of fluid momentum according to the Newtonian principle,

$$F = \frac{d(mV)}{dt}$$

Any two systems of flow—which may be designated by any subscripts, such as A and B—if already geometrically similar, will also be dynamically similar if all corresponding forces acting in the two systems are in the same ratio as the existing inertial qualities, designated by I (corresponding to the right-hand term in the Newtonian equation). Specifically, for dynamic similarity,

$$\frac{F_{g_A}}{F_{g_B}} = \frac{F_{v_A}}{F_{v_B}} = \frac{F_{\sigma_A}}{F_{\sigma_B}} = \frac{I_A}{I_B}$$

Expressing each of these terms in general dimensional units,

$$F_g = L^3\gamma;\quad F_v = L^2\mu\,\frac{V}{L};\quad F_\sigma = L\sigma;\quad I = \frac{L^3\rho V}{t} = L^2\rho V^2$$

substitution in these equations will yield the following algebraic statements for dynamically similar conditions:

Gravity : $$\frac{L_A^3\gamma_A}{L_B^3\gamma_B} = \frac{L_A^2\rho_A V_A^2}{L_B^2\rho_B V_B^2} \text{ and } \frac{V_A^2}{g_A L_A} = \frac{V_B^2}{g_B L_B} \quad [1]$$

Viscosity :

$$\frac{L_A\mu_A V_A}{L_B\mu_B V_B} = \frac{L_A^2\rho_A V_A^2}{L_B^2\rho_B V_B^2} \text{ and } \frac{L_A\rho_A V_A}{\mu_A} = \frac{L_B\rho_B V_B}{\mu_B} \quad [2]$$

Surface tension :

$$\frac{L_A\sigma_A}{L_B\sigma_B} = \frac{L_A^2\rho_A V_A^2}{L_B\rho_B V_B^2} \text{ and } \frac{L_A\rho_A V_A^2}{\sigma_A} = \frac{L_B\rho_B V_B^2}{\sigma_B} \quad [3]$$

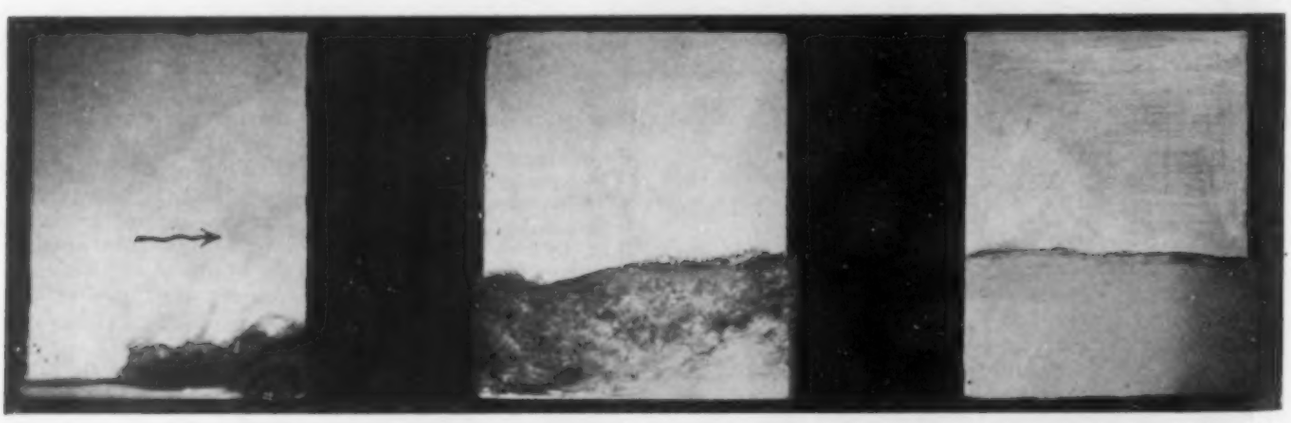
Expressions 1, 2, and 3 will be found to be dimensionless, or pure numbers, being known simply as **F, R,** and **W,** the Froude, the Reynolds, and the Weber number, respectively. That is:

$$\mathbf{F} = \frac{V^2}{gL} \text{ or } \frac{V}{\sqrt{gL}}$$

$$\mathbf{R} = \frac{VL\rho}{\mu} \text{ or } \frac{VL}{\nu}$$

$$\mathbf{W} = \frac{L\rho V^2}{\sigma} \text{ or } L\sqrt{\frac{\gamma}{\sigma}}$$

Thus, for dynamic similarity to exist among various systems of fluid motion, not only must all systems be completely similar geometrically, but the Froude, the Reynolds, and the Weber number must have the same value, respectively, in every system. Obviously the magnitude of the numbers will be independent of dimensional systems, since for a given set of conditions they will vary only with the arbitrary selection of the length parameter. This length term must, however, represent some dimension directly influencing the state of flow; that is, the Reynolds number for pipes must be a function of the diameter, not of the length. Similarly, the Froude number for open channels must vary with depth, not width.



A MODEL STUDY OF THE HYDRAULIC JUMP
Conducted at Columbia University to Determine the Length of the Jump as a Function of the Froude Number

Originally the Reynolds number was applied to flow under pressure in smooth circular pipes, all of which, if sufficiently smooth, are similar geometrically. The Froude number applies in general to problems of rapid transition with free surface, as explained by B. A. Bakhmeteff and A. Matzke in the article, "On Dynamical Similarity in Open Channel Flow, Particularly with Regard to the Hydraulic Jump," in Preprinted Papers, Aeronautic and Hydraulic Divisions, American Society of Mechanical Engineers (June 1934). Generally, however, two or three forces act simultaneously, and in the latter case, unless the action of two of the forces can be ignored, the third cannot be taken as the criterion for dynamic similarity. Moreover, complete geometrical similarity is difficult to achieve. In the case of pipe flow, roughness of different magnitude and form entails at once dynamical dissimilarity through geometrically dissimilar conditions. Froude's law as applied to ship resistance is not fully correct, since skin friction plays a significant part. Jets and weir nappes are influenced primarily by gravity, but also by viscosity and capillarity to a varying degree. As a result, unless one be content with deliberate approximations, it is almost impossible to attain complete dynamic similarity between any two systems of flow unless the scales are the same; the case of flow under pressure in smooth circular pipes is the only known exception to this unfortunate complexity of nature.

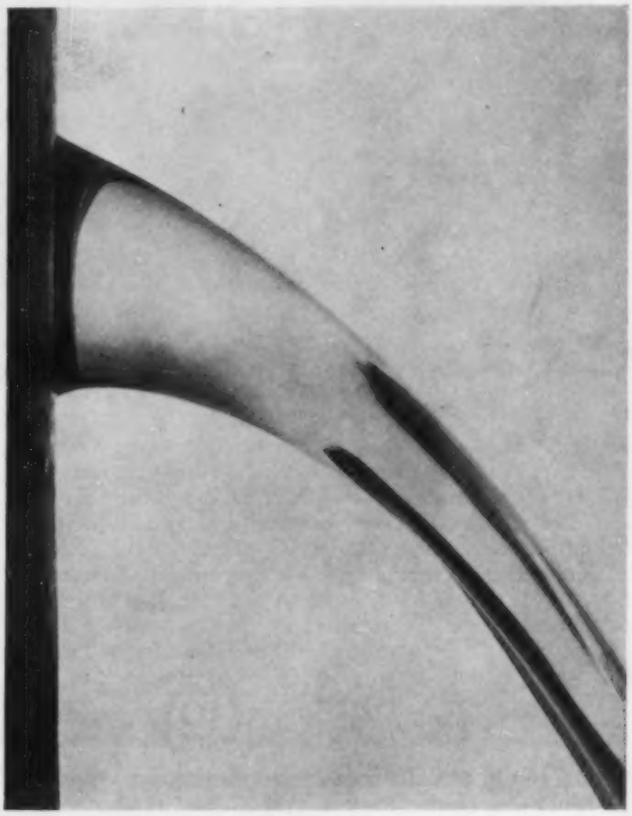
Apparently not daunted by this obstacle in the path of model research, many hydraulic experimenters not only attempt to surmount it blindly, but unknowingly create new barriers of their own making, even to the invention of incorrect dimensionless quantities. The Reynolds number has been applied promiscuously to rough pipes, Venturi meters, orifices, open channels, weirs, spillways—with viscosity apparently the sole criterion of variation in coefficient. Froude's law and that of Weber, on the contrary, through lack of the same publicity, have both failed to receive their just due in many cases and at the same time have been spared the embarrassment of frequent misapplication.

A few of the most publicized attempts of this sort may be mentioned as illustrations. For example, coefficients of the standard orifice have been plotted against the Reynolds number alone. Obviously the coefficient for orifice discharge under high heads is a function of viscous and capillary action together, gravity acting only on the free jet. But once the head becomes small, geometrical similarity no longer exists, and the gravitational effect on the jet is no longer negligible. Yet one and the same graph has been used to include all these conditions.

This same approach has now been extended to the V-notch weir. Although gravity is the principal influence in discharge, with surface tension and viscosity playing minor rôles, the coefficient is again plotted against the Reynolds number. Moreover, the length and velocity parameters are both made functions of the head, and hence are not independent variables.

Pipe orifices and Venturi meters, to be sure, are free from gravitational and capillary action and apparently lend themselves to treatment according to the Reynolds number. But geometrical conditions, not only in the form and throat ratio of the meter and the contraction ratio of the orifice, but particularly in the location of the piezometer connections, continue to prove stumbling blocks to investigators.

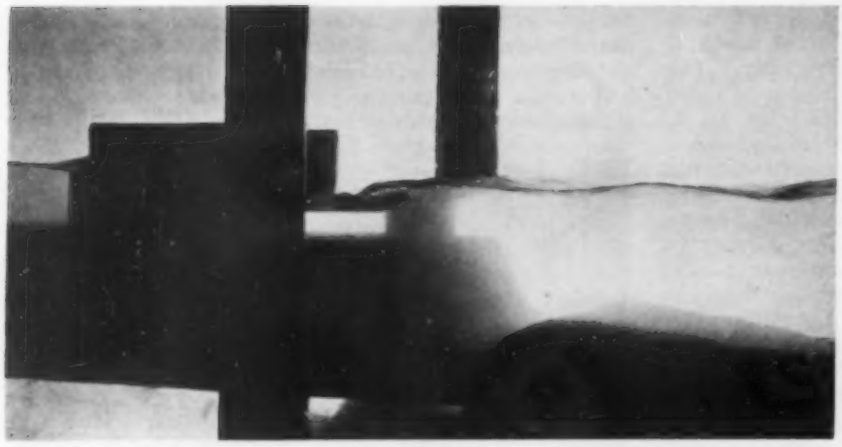
The success of plotting the frictional coefficient against the Reynolds number for circular pipes has naturally led to a similar attempt for open channels. For the length parameter, the hydraulic radius at once suggested itself, not only because of its constant use in the Chézy formula, but also because its application in pipe studies apparently had shown its reliability. However, since all circular pipes are similar geometrically, except for variations in roughness, this hydraulic radius bears a constant ratio to the diameter. On the other hand, it is obvious that a given hydraulic radius may signify an infinite variety of channel cross sections. Hence the use of such a Reynolds number for open channels ignores at the outset a fundamental requisite for dynamic similarity—exact similarity of geometric proportions—and appears to cover in one sweep every possible type of section.



ORIFICE DISCHARGE UNDER A VERY LOW HEAD

Illustrating the Unsymmetrical Silhouette of the Jet in the Region of Contraction. Here the Discharge Coefficient Is a Function of the Geometrical Proportions and of the Froude, Reynolds, and Weber Numbers. Experiment Conducted at Columbia University

Dimensionless numbers can be used effectively only by realizing the number of variables of which a phenomenon is a function and by taking the necessary steps to investigate each independently. A sharp-crested weir, for instance, discharges not only under the action of gravity, viscosity, and surface tension, but also according to geometrical conditions of channel and weir—that is, according to the ratio of head to weir height, width, crest thickness, roughness, and velocity distribution of the approaching flow. This fact has been brought out by the late Fr. Eisner in his "Ueberfallversuche in verschiedener Modellgrösse," Mitteilungen der Preussischen Versuchsanstalt für Wasserbau und Schiffbau (Berlin, 1933). Any attempt to compare the variation of the weir coefficient with viscosity and capillarity will be inaccurate without due consideration of each factor independently. It is pertinent that only a three-dimensional graph will suffice—that is, C against **R** and **W**—



A MODEL INVESTIGATION OF SCOUR BELOW SLUICE GATES

Froude's Gravitational Law Taken as the Sole Criterion of Similitude. Viscous and Capillary Effects Were Seldom Heeded in Such Investigations. From a Test in the Rehbock Laboratory at Karlsruhe

and that this will be general only if strict geometrical similarity has been attained in every point recorded. In other words, every change in head requires a corresponding change in every other geometrical dimension, including roughness. In this problem the Froude number may be varied only by a geometrical change—by varying the ratio of head to weir height—which entails further complication.

One of the best empirical relationships for a "universal" weir yet devised was developed by Rehbock in 1913,

$$Q = \frac{2}{3}\left(0.605 + \frac{0.08\,h}{Z} + \frac{1}{1{,}000\,h}\right)\sqrt{2g}\;Bh^{3/2}$$

This formula actually contains terms that are equivalent to functions of the Froude and Weber numbers. The expression within the parentheses is the discharge coefficient, the first term of which, as explained by Eisner, is a constant approximately equal to the limiting coefficient $\left(\frac{\pi}{\pi+2} = 0.611\right)$ for an infinitely high weir discharging under the action of gravity alone. The second term, taking the square root of the head to indicate velocity and the weir height Z as the length parameter, is really a constant times the Froude number, that is,

$$0.08\,\frac{h}{Z} = \text{constant}\;\frac{V^2}{gZ}$$

The third term, $1/1{,}000\,h$, is not dimensionless because it is expressed in definite metric units, but it is equivalent numerically to a constant divided by **W**:

$$\frac{\text{constant}}{h\sqrt{\frac{\bar{\gamma}}{\sigma}}}$$

It is obvious that the formula includes no mention of the Reynolds number and hence ignores viscous influences; furthermore, the width is assumed to have no bearing on the conditions of flow. Yet for an admittedly empirical formula, it is interesting to note the unconscious inclusion of two of the three dimensionless numbers for incompressible fluids.

Hydraulic phenomena are so complex that one can readily understand why the hydraulician often turns to empirical equations for the solution of practical problems. However, if greater accuracy is sought by the analytical method, this very complexity should be a warning against the acceptance of the dimensionless number as a cure-all for every theoretical ill.

As an outstanding example of the dangers attending the general application of dimensionless quantities in the interpretation of flow phenomena, attention is called to the Venturi flume with either floor or side contractions, a well-known form of discharge meter both in America and abroad.

This type of hydraulic device is used as an illustration not only because of its general use, but also because it presents a most complex problem owing to the variable form of the free surface. The Venturi flume causes a rapid acceleration of flow at the abrupt contraction of section, followed by a more gradual deceleration in the diverging region beyond the throat. The discharge can then be written as a function of the meter dimensions and the depths at the inlet and at the throat. Experiments have shown that this expression must also include a coefficient, depending presumably on losses of energy.

Elementary principles of hydrodynamics will suffice to show that either rapid acceleration or deceleration in non-uniform flow results in curvilinear motion; moreover, curvilinear motion is always accompanied by a

VORTEX MOTION AT THE BASE OF A SHARP-CRESTED WEIR

From Author's Experiments Conducted at the Massachusetts Institute of Technology River Hydraulic Laboratory. Discharge Coefficients Are Assumed to Be a Function of Gravitational and Capillary Action and Vertical Geometrical Proportions; Yet Such Turbulent Formation as Shown Above Results from Frictional Loss Along Walls and Floor and Hence Depends also on Kinematic Viscosity, Width, and Roughness of Channel. Photograph, Courtesy of the Massachusetts Institute of Technology

departure from static pressure conditions—that is, pressure head is no longer a linear function of depth below the free surface. Now it is customary in using the Venturi flume for measuring purposes to rely on piezometers at inlet and throat to indicate depth. As a matter of fact, there are usually several floor and wall piezometers (all interconnected) at a single section immediately before the contraction, and a similar device at a single section of the throat. Hence not only is there a possibility of discrepancy in any single measurement of depth because of non-static conditions, but the change in form and position of the surface curve with changing discharge would tend to make this discrepancy of variable magnitude. While such measurements are entirely satisfactory for determining discharge with the one meter in question, considerable doubt must accompany their use in prophesying the behavior of fluids in general nonuniform flow.

Through proper variation of upstream and downstream depths, the Venturi meter may be made to discharge with a hydraulic jump in the diverging section. It often happens that this jump may form when the Froude number computed for the throat section is still less than unity, while reference to the previously mentioned article by Bakhmeteff and Matzke on the hydraulic jump will show that the Froude number is always greater than unity for flow above the critical velocity. In order to account for such apparent inconsistency, a new, but erroneous, dimensionless quantity has been devised, which some experimenters have called the "Boussinesq number," and which, for certain relatively narrow flume proportions, is greater than unity when the jump occurs. Hence, they have reasoned, the hydraulic jump can form in any open channel only when the "Boussinesq number" is greater than unity, and proponents of the Froude criterion must therefore be in error.

This "Boussinesq number" is identical with the Froude number for open-channel flow

$$\mathbf{F} = \frac{V}{\sqrt{gL}} = \sqrt{2\,\frac{\frac{V^2}{2g}}{d}}$$

with the exception that twice the hydraulic radius is substituted for the usual length parameter, thus giving for a rectangular channel

$$\frac{V}{\sqrt{g_2\,\mathbf{R}}} = \sqrt{2\,\frac{\frac{V^2}{2g}}{\frac{2Bd}{2d+B}}}$$

in which d, B, and R are respectively depth, breadth, and hydraulic radius of the flow section. That is, the number is no longer a direct indication of the ratio between kinetic energy and potential energy, involving as it does an elastic relation between width and depth of flow. For only one ratio of depth to width—that is, when the depth is half the width—will the Froude and "Boussinesq" numbers be equal; and while the Froude number for a given velocity head and depth of flow will be constant regardless of width, the "Boussinesq number" will be different for every width and hence will apparently indicate that the gravitational phenomenon is dependent on the width as well. This is obviously an erroneous deduction.

OUTDOOR EXPERIMENTS ON MOVEMENT OF GESCHIEBE IN RIVERS

Conducted at the Forschungsinstitut für Wasserbau und Wasserkraft in Bavaria, Germany, near Walchensee. Laboratories Are Now Considering the Influence of Viscosity in Such Studies

The faults in such an analysis are of the utmost importance, as they illustrate a frequent misunderstanding of non-uniform flow—in particular of curving flow in regions of rapid transition. A dimensionless number computed for conditions at one section applies correctly to that section alone. If the flow is converging or diverging, not only must pressure change due to curvature be taken into account in measuring the depth and the specific energy of flow, but one must realize as well that the magnitude of the dimensionless number will also change from section to section with the variation in velocity and depth, because of the changing area of the flow section.

That the jump may occur in the diverging part of the Venturi flame is sufficient indication that the Froude number rises above unity at some point beyond the throat. On the other hand, the fact that the "Boussinesq number" might give satisfactory curves for the meter in question is neither reason to doubt a correct physical relationship incorrectly used nor ground to apply an obviously elastic number to general flow problems in which its elasticity would prove its downfall.

If one really desires to determine the effect of variation

in width upon discharge conditions (that is, viscous effects as distinguished from geometrical), one must treat the problem strictly in accord with Reynolds' law of dynamic similarity, involving the independent variation of only width, velocity, and kinematic viscosity. The "Boussinesq number" will be of no service to the investigator in this respect.

There are rumors of the acceptance of this new dimensionless quantity by various hydraulicians on the two continents. As a warning, it is worth repeating a simple rule of dimensional analysis: If an expression is not dimensionally homogeneous, it is surely wrong; if an expression actually is dimensionally homogeneous, it may possibly be right.

Dimensional Analysis in Model Studies

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SOME knowledge of dimensional analysis should certainly form a part of the equipment of the hydraulic investigator although this of course should not be the sum total of his mathematical equipment. The whole field of hydromechanics should be open to him, but unfortunately progress is often barred by a lack of mathematical knowledge.

For a proper background the young engineer specializing in hydraulic models should read a few of the older papers on the subject, some of which might well be taken as models in the clear presentation of new and complicated ideas. The recommended list on page 570, is confined to papers in the English language. These are arranged in chronological order. No reference to Lord Rayleigh has been given solely because it is so difficult to select one or two of his papers as outstanding among the many in which he made use of dimensional reasoning. Many other great men of science have considered this subject worth writing about, such as Helmholtz, Planck, and Sir Oliver Lodge, to say nothing of contemporary scientists like Lewis, Tolman, Eddington, Campbell, and Bridgman. After a background has been laid from the articles listed, the study of several of the systematic presentations of dimensional analysis shown in the list on page 571 is urged.

If the young engineer were to read all these authorities, he would find that there is much disgreement among them as to the fundamental principles of dimensional analysis and as to the best way of applying them, but he would gain some idea as to how dimensional analysis might help him in his practical problems. Skill in using it of course can come only with practice.

AN EXPOSITION OF THE THEORY ACCORDING TO BUCKINGHAM

Turning to a typical case of pipe flow, assume that the slope, S, of the hydraulic gradient depends on the following five factors, and these only, the dimensions of the quantities at the left being given in brackets at the right:

D	= diameter of pipe	$= [L]$
V	= mean velocity over the section	$= [LT^{-1}]$
ρ (rho)	= density of the fluid	$= [ML^{-3}]$
μ (mu)	= viscosity of the fluid	$= [ML^{-1}T^{-1}]$
g	= acceleration of gravity	$= [LT^{-2}]$

in which the negative exponents indicate "per"; T is time; M is mass; and L is length. The six variables may be connected by the following formula:

$$f(S, D, V, \rho, \mu, g) = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots [1]$$

These six quantities are all of different dimensions and involve three fundamental units. Then, according to Buckingham's π theorem, the quantities may be arranged into three independent dimensionless groupings, π_1, π_2, and π_3; and the formula may be written as

$$\phi(\pi_1, \pi_2, \pi_3) = 0, \text{ or } \pi_3 = \Phi(\pi_1, \pi_2) \quad \ldots\ldots\ldots [2]$$

in which ϕ and Φ each indicate a "function of." Since S is dimensionless, it can be taken as one of the dimensionless arguments, say π_3. Then π_1 and π_2 are obtained by taking any one of the remaining quantities to a known power (usually the exponent is taken as 1, but in what follows it is taken as -1) and three others (which between them involve all the different fundamental units) to unknown powers x, y, and z. (The question is sometimes raised as to why these π's must be of this form, and not of some other, such as $\pi_1 = e^D \sin V \log \rho g$. The explanation may be found by referring to Bridgman's book. It should be obvious, however, that a dimensional quantity cannot be an exponent or the argument of a logarithm or of a trigonometric function. But the dimensionless π's, when determined, might be in any of these forms.) These unknown exponents are then determined by the requirement that since the whole expression is dimensionless, the exponents of each fundamental unit must add up to zero. Take

$$\pi_1 = D^{x_1} V^{y_1} \rho^{z_1} g^{-1} \text{ and } \pi_2 = D^{x_2} V^{y_2} \rho^{z_2} \mu^{-1}$$

Then by substituting the dimensions of the quantities,

$$[L^{x_1}][LT^{-1}]^{y_1}[ML^{-3}]^{z_1}[LT^{-2}]^{-1} = [L^0M^0T^0]$$

Equating the exponents of T, $-y_1 + 2 = 0$, or $y_1 = 2$. Equating the exponents of M, $z_1 = 0$. Equating the exponents of L, $x_1 + y_1 - 3z_1 - 1 = 0$, $x_1 = -1$, and $\pi_1 = \dfrac{V^2}{gD}$ which is the Froude number, **F**. If the hyphenating method were carried to its extreme this might be called the Newton-Bertrand-Reech-Froude-Helmholtz number, because all these men had a part in bringing the idea of this dimensionless criterion to its present form.

Similarly, $[L]^{x_2}\,[LT^{-1}]^{y_2}\,[ML^{-3}]^{z_2}\,[ML^{-1}T^{-1}]^{-1} = [L^0M^0T^0]$

Again equating the exponents of T, $-y_2 + 1 = 0$, or $y_2 = 1$

Equating the exponents of M, $z_2 - 1 = 0$, or $z_2 = 1$

Equating the exponents of L, $x_2 + y_2 - 3z_2 + 1 = 0$ or $x_2 = 1$, and

$\pi_2 = \frac{DV\rho}{\mu}$. Since $\frac{\mu}{\rho} = \nu$ = kinematic viscosity, then $\frac{DV\rho}{\mu} = \frac{DV}{\nu}$, which is the Reynolds number, **R**.

Then $$S = \Phi(\mathbf{F}, \mathbf{R}) \quad [3]$$

Dimensional analysis does not tell us what the form of the function will be; that must be determined by experiment. If, for example, two sets of values are taken so that **F** is the same in each case, but **R** is twice as much in one case as in the other but in both cases well above critical, it will be found that S is not very much different in the two cases. But if the two sets are selected so that **R** is the same in each case, but **F** twice as much in one case as in the other, S is also found to be just twice as much. That is, experiment shows that Equation 3 is of the form:

$$S = \mathbf{F}\psi(\mathbf{R}) = \frac{V^2}{gD}\psi(\mathbf{R}) = \frac{fV^2}{2gD}, \text{ where } f = 2\psi(\mathbf{R}) = \theta(\mathbf{R}) \quad [4]$$

in which ψ and θ each indicate an additional "function of." The form of this function has been rather carefully studied and reported during the last twenty years by a number of investigators, particularly by Blasius, in *Handbuch der Physik*, Vol. 7 (1927), page 140; by Hopf, in the same book, page 141; by Biegeleisen and Bukowski, in "Grundlagen zur Berechnung von Wasserrohrleitungen," *Gesundheits-Ingenieur*, Vol. 37 (1914), pages 477–495, 554–560, 593–601; by Charles H. Lees, in *Proceedings of the Royal Society of London*, Series A, Vol. 91 (1914), pages 46–53; and by Linquist, "Om Modellregler Eller Likformighetssatser vid Vattenbyggnadstekniska Försök," *Teknisk Tidskrift*, Nos. 30, 34, 43 (Stockholm, 1925). Emory Kemler has collected most of the pertinent data for many kinds of pipe in graphical plottings of f against **R**, and his data are published in "A Study of the Data on the Flow of Fluids in Pipes," *Transactions of the American Society of Mechanical Engineers*, Vol. 55 (HYD) (1933), pages 7–32.

REYNOLDS' METHOD

Perhaps a word should be said as to the difference between this method and that used by Osborne Reynolds. By using rate of loss of pressure instead of S and g, he reduced the number of quantities to five. He then assumed that one of these variables varied as the product of certain unknown powers of the other four. The requirement of dimensional homogeneity gives three equations, so that the unknown exponents can all be expressed in terms of one unknown. When the terms involving this unknown are collected on one side of the equation, and those with known exponents on the other, **F** and **R** appear. The method gives the same result as Buckingham's if the function is a simple exponential function, but it is very much more restricted, and the assumption that the function is of this form may not be correct.

APPLICATION TO OPEN CHANNELS

The analysis given also applies to flow other than that in pipes, in which case D might have been replaced by any other length. Thus it might have been the hydraulic radius of a uniform channel. Then, by exactly the same reasoning, S will be a function of **F** and **R**, but perhaps not the same function as for a pipe. Or, if unwilling to assume that the hydraulic radius tells the whole story, let D be the depth and W the width. Then W/D will equal r_1 and S will be a function of **F**, **R**, and r_1. If the shape of the channel is so complicated as to require three or more dimensions to describe it, D can be taken as any one dimension, and the other dimensions will be r_1D, r_2D, r_3D, and so on.

If a slope had not been involved in the problem, and only the five dimensional quantities had been considered, the equation would have been

$$\phi(\mathbf{F}, \mathbf{R}) = 0, \text{ or } \mathbf{F} = \Phi(\mathbf{R}) \quad [5]$$

If the dimensions involved are small enough so that capillary forces become appreciable, the surface tension, σ, of the fluid becomes another of the quantities that must be considered, and an additional relationship is obtained as shown by

$$\pi_4 = \frac{V^2D\rho}{\sigma} = \mathbf{W}, \text{ the Weber number.}$$

Or if the velocities increase until they become of the order of the velocity of sound in the fluid, the bulk modulus of elasticity of the fluid, K, becomes one of the quantities that must be considered, and the relationship becomes

$$\pi_5 = \frac{V^2\rho}{K} = \mathbf{C}, \text{ the Cauchy number.}$$

These five dimensionless numbers are the only ones ordinarily used in hydraulics, although of course they do not begin to exhaust the possibilities of dimensionless groupings of these seven quantities. Any function of a dimensionless number is also dimensionless, and any function of two or more such numbers is dimensionless. Thus either **F** or **R** could be replaced by $\frac{\mathbf{F}}{\mathbf{R}^2} = \frac{\nu^2}{gD^3}$.

APPLICATION OF THE THEORY TO MODELS

Applying this theory to model tests, it is evident that if variations in the viscosity of the fluid do not affect the problem, **R** cannot enter into the result. Then, assuming that variations in surface tension and modulus of elasticity also have no effect, everything depends upon **F**. The model must have the same **F** as the prototype. Since g is the same in both model and prototype, V will have to vary as the square roots of the dimensions. This fact has been known for more than fifty years and is the cornerstone on which all hydraulic model computations are founded.

But in most cases large variations in the viscosity of the fluid have an appreciable effect on the other quantities. Then, for exact representation, both **F** and **R** will have to be the same in the model as in the prototype. This means that since V_m will be $\frac{V_n}{\lambda^{0.5}}$, ν_m will have to be $\frac{\nu_n}{\lambda^{1.5}}$, where λ is the ratio of the dimension in nature, D_n, to the dimensions in the model, D_m. If the scale of the model is 1:25, this will require that the fluid used in the model have a kinematic viscosity, ν, of only $^1/_{125}$ of that of the prototype. Thus it is possible to completely represent the action of a very viscous fluid by a small-scale model using a less viscous fluid. But

unfortunately water, the fluid most often represented in the model, is one of the least viscous of fluids.

Table I gives the kinematic viscosities of several fluids. Mercury has the smallest kinematic viscosity, due to its high density, but at ordinary temperatures it is only about eight times as fluid as water, so that the proper scale ratio for complete similarity would be only 1:4. As it would ordinarily be more expensive to perform a test on a 1:4 model with mercury, than to test the full-size structure with water, complete similarity seems to be an impracticable ideal in this case.

There is of course another way out, as has been suggested by Alton C. Chick and probably others. If the model could be rotated rapidly, a centripetal acceleration could be produced which would be many times as much as g. Then if the model scale were 1:20, and mercury were used at a temperature such that its kinematic viscosity were only 0.1 that of water, the velocity in the model would have to be only twice as great as that in nature to keep the Reynolds number the same as in nature. The Froude number could be made the same by using an acceleration 80 times that in nature. But the difficulty of rotating the model, circulating pump, and other necessary apparatus, and making any measurements at the high speed that would be required, seems to remove this method from the realm of possibility.

TABLE I. KINEMATIC VISCOSITIES OF VARIOUS FLUIDS AT VARIOUS TEMPERATURES

In Square Millimeters per Second

FLUID	0 C = 32 F	20 C = 68 F	50 C = 122 F
Mercury	0.1235	0.1145	0.1032
Carbon bisulfide	0.332	0.290	.....
Ether	0.386	0.327	.....
Chloroform	0.458	0.378	0.296
Carbon tetrachloride	0.827	0.608	0.426
Benzol (benzene)	1.002	0.738	0.516
Water	1.794	1.010	0.556
Alcohol (ethyl)	2.199	1.521	0.919
Air	13.22	15.01	17.86
Castor oil	.....	1,027.0	100.0
Mexican petroleum	.....	5,100.0	450.0

Fortunately, in most cases which the hydraulic engineer wishes to investigate, the effects depend principally on the Froude number, and only secondarily on the Reynolds number. And moreover, it seems generally true that as the Reynolds number becomes large, the function involving it approaches an assymptote. So that if a 1:100 model is tested, then a 1:50 model, and finally a 1:25 model, for example, generally a very accurate forecast of the values that would apply to the full-size structure can be obtained. This method has been used quite extensively in aeronautical design.

It should be remembered that the region extending from approximately **R** equals 1,000 to **R** equals 5,000 is one of instability, in which the flow changes from laminar to turbulent, and that the model and the prototype must both stay on the same side of this region. This generally determines the minimum size of model that can be used. The largest size is determined by considerations of cost, laboratory space, and water supply. In the case of tests that involve a time interval, such as those for erosion, sedimentation, and the like, another advantage of the small model is the shorter time required for the test.

Considering again flow in pipes, one variable that was omitted was the roughness of the inner surface. What are the dimensions of roughness? The obvious answer is that it has length, that is, the average height of the projections which produce the roughness, or some such quantity. Then the ratio of this distance to the diameter of the pipe would be another dimensionless number that would enter the formula. This point of view is developed more at length in the articles by Gibson in *Engineering*, in that by Chick in Freeman's *Hydraulic Laboratory Practice*, and in my discussion of Vogel's paper in PROCEEDINGS, all referred to in the box on page 571. But if this is the proper argument to use regarding roughness, I now believe that it does not enter the formula to any single power. This whole subject merits further study.

Roughness may be looked at from another point of view. Manning's formula seems to represent the tests on uniform open channels fairly well. Solving it for the coefficient of roughness, n, gives

$$n = \frac{1.486\, R^{0.67}\, S^{0.50}}{V}.$$

Then the dimensions of n are $[n] = \frac{[L]^{0.67}}{[LT^{-1}]} = [L^{-1/3}T]$.

If this is put into the problem as another of the dimen-

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sional quantities upon which the slope depends, a new dimensionless argument is obtained, which can be put in the form, $\frac{ng^{0.5}}{D^{1/6}}$.

From the first point of view, the model should represent the prototype exactly to scale, including all the little projections and indentations which make up the roughness. If the scale ratio is 1:64, every projection in the prototype should be represented by a projection just $^1/_{64}$th as high in the model. From the second point of view, the coefficient of roughness (n) in the model should be the coefficient of roughness (n), in nature divided by the sixth root of the scale ratio. In the example of a 1:64 scale ratio, n in the model should be half of n in the prototype. Perhaps these two rules both say the same thing. But if so it remains to be proved. At any rate they both require that small-scale models should be very smooth, sometimes smoother than they can be made. This introduces another limit to the reduction in size of the model.

Both these limitations to small scales can be circumvented to a certain extent, at least in river work, by the use of distorted models. In most rivers the hydraulic radius is practically equal to the mean depth, and the model can be "narrowed up" considerably without this ceasing to be so. Then the limit imposed by the critical value of the Reynolds number applies only to the vertical scale, and the horizontal scale can be made much smaller. But this has the effect of increasing the slope, so that the water runs too fast unless the model is made rougher than the rules just stated would require. Thus distortion has its advantage, whatever its drawbacks from some points of view, and very neatly kills two birds with one stone.

Scale ratios for various quantities involved in the flow in distorted models of open channels based on Manning's formula are given in Table II. Each lower case letter indicates the ratio of the quantity represented by the corresponding capital letter in the prototype to the corresponding quantity in the model. In it λ is the horizontal scale ratio; h is the vertical scale ratio (ratio of height in the prototype to height in the model); g is the ratio of the acceleration of gravity in the prototype to the corresponding acceleration in the model; j is the ratio of the weight per unit volume of the fluid in the prototype to the corresponding value in the model (both of these would ordinarily be unity), and n is the ratio of Kutter's n in the prototype to Kutter's n in the model. The next to the last column gives the corresponding scale ratio in the ideal case, in which viscosity is neglected, there is no distortion, and **F** is the same in model and prototype. The last column gives the value of n required to make the two other columns agree, that is, to keep **F** in the distorted model equal to the value in the prototype. If in the first column h is given as equal to λ then a more accurate conversion ratio for undistorted models of channel flow is obtained than that given in the second column.

The results here given can be reached by quite different lines of reasoning, and they are of course not original with me. No attempt will be made to list all the sources from which I have gleaned. But one from whom I received much knowledge and inspiration should be mentioned, Floyd Nagler, M. Am. Soc. C.E., by whose untimely death last autumn our profession suffered a very serious loss.

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TABLE II. SCALE RATIOS FOR MODEL DIMENSIONS

QUANTITY	SCALE RATIO FOR DISTORTED MODEL	IDEAL SCALE RATIO	VALUE OF n REQUIRED TO MAKE THEM AGREE
Length and breadth	λ	λ	.....
Height	h	λ	.....
Horizontal area	$a_h = \lambda^2$	λ^2	.....
Vertical area	$a_v = \lambda h$	λ^2	.....
Volume	$a_v\lambda = a_h h = \lambda^2 h$	λ^3	.....
Weight	$w = j\lambda^2 h$	$j\lambda^3$	.....
Unit pressure	$p = w/a_h = jh$	$j\lambda$	.....
Slope	$s = \lambda^{-1}h$	1	.....
Mass	$m = w/g = jg^{-1}\lambda^2 h$	$jg^{-1}\lambda^3$	.....
Velocity	$v = \lambda^{-0.5}h^{1.17}n^{-1}$	$g^{0.5}\lambda^{0.5}$	$\frac{h^{1.17}}{g^{0.5}}\left(\frac{h}{\lambda}\right)^2$
Discharge	$q = Va_v = \lambda^{2.5}h^{2.17}n^{-1}$	$g^{0.5}\lambda^{2.5}$	$\frac{h^{2.17}}{g^{0.5}}\left(\frac{h}{\lambda}\right)^2$
Time	$t = \lambda/v = \lambda^{1.5}h^{-1.17}n$	$g^{-0.5}\lambda^{0.5}$	$\frac{h^{0.17}}{g^{0.5}}\left(\frac{h}{\lambda}\right)$
Acceleration	$v/t = \lambda^{-2}h^{2.33}n^{-2}$	g	$\frac{h^{0.17}}{g^{0.5}}\left(\frac{h}{\lambda}\right)$
Force	$f = jg^{-1}h^{2.33}n^{-2}$	$j\lambda^3$	$\frac{h^{0.17}}{g^{0.5}}\left(\frac{h}{\lambda}\right)^{1.50}$
Momentum	$mv = jg^{-1}\lambda^{1.5}h^{2.17}n^{-1}$	$jg^{-0.5}\lambda^{3.5}$	$\frac{h^{0.17}}{g^{0.5}}\left(\frac{h}{\lambda}\right)^2$
Energy and work	$f\lambda = jg^{-1}\lambda h^{2.33}n^{-2}$	$j\lambda^4$	$\frac{h^{0.17}}{g^{0.5}}\left(\frac{h}{\lambda}\right)^{1.50}$
Power	$p = e/t = jg^{-1}\lambda^{-0.5}h^{4.5}n^{-3}$	$jg^{0.5}\lambda^{4.5}$	$\frac{h^{0.17}}{g^{0.5}}\left(\frac{h}{\lambda}\right)^{1.33}$

Electrolysis and Corrosion Damage Warehouse

Repair of Reinforced Concrete Structure Injured by Hide-Curing Salts and Acids

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AFTER two years of use for hide-curing purposes, a reinforced concrete warehouse built in 1925 developed leaks in the first-floor slab, making it necessary to waterproof the floor. Five years later pieces of concrete began to fall off the columns and basement walls revealing a most serious condition of corrosion of the reinforcing steel, which threatened the integrity of the building. A complete investigation showed that the curing brine seeping through the concrete had reacted, probably with impurities in the steel, to form minute galvanic cells. Hydrochloric acid formed by electrolytic action had in some cases nearly destroyed the steel without leaving any marks of such destruction on the surface of the concrete. The hidden damage due to electrolysis was more serious than the obvious damage due to corrosion. Mr. McCreery not only describes this startling damage but also the steps taken to stop it and return the building to complete usefulness.

OCCASIONALLY an engineer is called upon to examine and report on a building after a failure. Seldom is one privileged to determine the cause and extent of damage before failure occurs. In the case of the Class A warehouse of a hide merchant in Los Angeles, the engineers were accorded the almost unique privilege of diagnosing the ills of a reinforced concrete building still in service, making the necessary repairs, and returning the structure to its original condition. This two-story and basement building of flat-slab design was erected in 1925 for hide-curing purposes. The fresh hides, with layers of salt between, are stacked on the floor in piles for 30 days or longer and allowed to cure. One product of this process is a nearly saturated solution of salt containing a small percentage of weak acids. This solution is allowed to run on the floors to drains and thence to the sewer. Because it is necessary to have a moist atmosphere in the warehouse, the slope of the floors is very flat and the solution often stands in pools around the hides. Up to the present only the first floor and basement have been subject to such use.

That there was a need of special waterproofing was apparent about two years after the building was occupied, when leakage through the first floor slab at construction joints and expansion cracks became so severe that the owner put an asphalt mastic covering on the first floor. At the same time—more for the sake of appearance than for waterproofing—the columns and walls of the first floor and basement were painted to a height of 5 ft 3 in. with a black coal-tar paint.

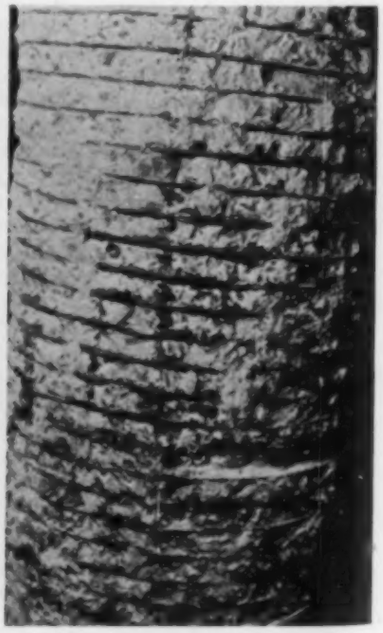
Indications of damage to the building became alarming in the spring of 1932, when large chunks of concrete spalled off the columns and walls in the basement. One of the illustrations shows a typical example of the spalling on a column. By the middle of the summer of 1932 all but 2 of the 40 free-standing columns in the basement had similar spalled places or were hollow to the point of incipient spalling.

Structural damage to the building was confined almost entirely to the reinforcing steel in the walls and columns of the basement and first floor and in the first-floor slab. The injury to the steel was of two distinctly different kinds. The only kind to give outward evidence of its presence was corrosion-oxidation of the steel, induced and accelerated by the salt water. This was the less destructive of the two kinds as far as the steel was concerned, but it gave immediate and unmistakable signs of its presence by disintegrating the concrete fireproofing.

By far the more serious of the two kinds of damage was that caused by electrolytic action on the steel. The effect of this process was greater than that of corrosion, yet it gave no outward sign. The mechanical disintegration of the concrete due to corrosion could be detected by tapping the concrete with a hammer long before spalling took place. In the case of the electrolytic damage, 95 per cent of the steel in a column or wall could be destroyed yet the concrete would ring true under the hammer, and there would be no visible sign of the condition inside. These two actions, corrosion and electrolysis, commonly took place together; the effects of the electrolysis were found under the rust film.

Some bars $1/2$ in. square taken from the basement wall

Fireproofing Spalled off Basement Column After Seven Years of Use



Loose Material Stripped and Fireproofing Chipped Away

REPAIR OF COLUMNS DAMAGED

are shown in another illustration. On them the effect of electrolytic action is apparent. There was practically no corrosion on these bars, and the concrete in the section of wall from which they were taken was entirely sound in other respects. It required real work with air hammers to remove it.

ELECTROLYTIC ACTION

The theoretic basis on which the diagnosis of electrolysis was based was that the salt and water penetrating the concrete to the steel combined with the impurities in the steel to set up a small galvanic cell. The concrete saturated with the salt solution made a comparatively good conductor, and the galvanic cell created a potential to the earth sufficient to induce a flow of current. This led to the generation of hydrochloric acid, which attacked and destroyed the steel. Tests made with a high-resistance voltmeter revealed a difference of potential between the steel and the earth of 0.1 v, which was remarkably constant throughout the building. Tests also showed that there was no leakage from the utilities entering the building; the highest potential difference registered in this group of tests was 1.5 mv.

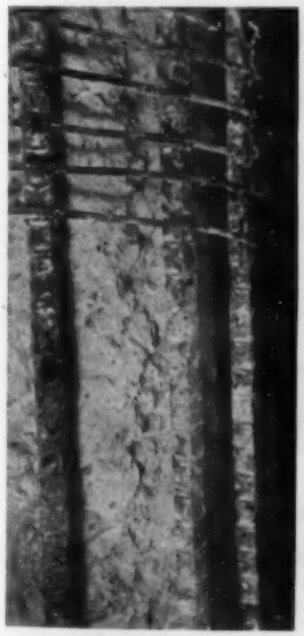
This electrolytic action left residues of two kinds. The most common was a black or gray gummy substance which filled pockets eaten in the steel. Chemical analysis of this substance revealed it to be largely chlorine. The other residue was a black, syrupy liquid about the consistency of molasses, which penetrated the concrete surrounding the bar affected and filled all accessible voids. On exposure to the air it very quickly oxidized to a hard, shiny material rust-brown in color.

Only in scattered instances could any decomposition of the concrete be detected. In a few places in construction joints and expansion cracks in the first-floor slab, where the liquid from the hides had a chance to leach through, there was a noticeable change in the character of the concrete, accompanied by considerable loss of strength. In these places the steel was badly eaten and corroded.

Two years after the building was occupied, when the columns both in the basement and on the first floor were painted with a coal-tar paint, it was too late to prevent the spread of the damage. Cracks had been made in the concrete fireproofing which the paint did not seal, so that the salt water had easy access to the steel. Also, because of this paint, capillary action drew the salt solution up the columns to the top of the painted area. For this reason the damage extended generally to a height of 5 ft, but in some cases the corrosion had advanced to the base of the column capital, a distance of 8 ft 6 in. above the floor line. To these heights the spirals in the columns were corroded to such an extent as to be useless, and the vertical bars were badly rusted to the same heights. In addition, the vertical bars were extensively pitted by electrolysis. Subsequent cleaning of the bars revealed many pits, which were covered with scale or rust. Examples were uncovered in which more than half the bar had been eaten away.

From the spread of the damage noted during the comparatively short time that the building was under our observation, it would appear that the work was started at a critical time. In the latter part of July 1932, when the first investigation of the building was made, six columns on the first floor were hollow or spalled. On October 27, 1932, when work was started on this floor, 43 of the 71 columns and pilasters gave unmistakable signs of corrosion. It is possible that the vibration from the cutting away of the concrete on the basement columns hastened the disintegration of the columns on the first floor, but for this to happen the corrosion would have had to reach the point of incipient spalling. The condition on the first floor was aggravated in 12 of the 40 columns by the fact that the steel was about $1^3/_4$ in. off center, thus bringing the longitudinal steel and the spirals almost to the face of the column on one side.

Damage to the steel in the walls was due more to electrolysis than to corrosion. Every linear foot of wall in the basement was affected for a height varying from 2 to 8 ft; the major part was damaged to a height of 5 ft 3 in., or to the top of the coal-tar paint. The basement wall was designed as a retaining wall, and the vertical bars were spaced 6 in. on centers. Every bar was affected to some extent, and some few bars were in very bad condition, having had as much as 95 per cent of their cross section eaten away. Because the electrolysis gave no outward sign of its presence, it was necessary to strip the concrete from the entire basement wall to make sure that all damaged steel was found and all sources of current destroyed. Neither in the walls nor in the columns in the basement did the damage extend more than $1^1/_2$ in. below the floor line.



Concrete Removed from Bars; Sodium Chloride in Black Spots

Finished Column with Waterproofing Added

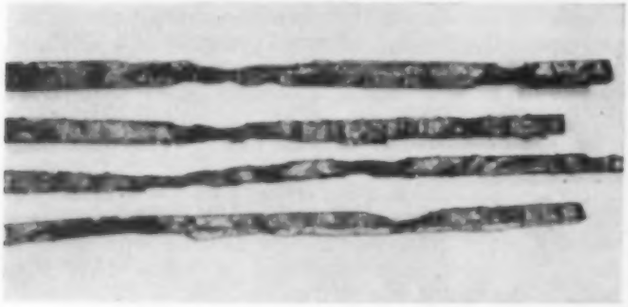
BY ELECTROLYSIS AND CORROSION

FIRST FLOOR SLAB SERIOUSLY WEAKENED

Serious as was the damage to the walls and columns that to the first-floor slab was by far the most serious and dangerous. There were two construction joints lengthwise of this building and three crosswise, dividing it into 12 pours. All these joints were unprotected during the first two years of use, and the owner has stated that through them salt water literally poured from the hides stored on the slab. Of course these joints were all in the midspan of the floor panels, so that the steel was attacked the quickest where it was needed the most. The fact that the mastic flooring stopped this leakage when it did, undoubtedly prevented a collapse of several floor panels if not of the entire structure. The steel was badly eaten and corroded where it crossed these construction joints. In a triangular section across the joint, the concrete had lost most of its strength. This is the only condition under which decomposition of the concrete

was found. The same facts apply to the expansion cracks in the slab, of which there were many at an angle with the steel.

Many of the floor panels had strain cracks on the under side at or near the center of the span. These were carefully watched, and the loads on the floor above were checked. Many cases of overloading were found, and these, together with the weakened condition of the steel, readily accounted for the cracking. In one case a load of 400 lb per sq ft was being carried on the slab, which was designed for 250 lb per sq ft. The underside of this section of slab was generously covered with cracks. In many panels the steel was found to be from 2 to 4 in. out of place, which was a further cause of weakness.



EFFECT OF ELECTROLYTIC ACTION ON BASEMENT WALL STEEL
Steel Reinforcing Bars Are 1/4 In. Square

Late one Thursday afternoon, when a crack was observed on the center line of one panel carrying the allowable load of 250 lb per sq in., a pencil mark was made at each end of the crack. By 9 a.m. of the following Saturday the crack had progressed 6 in. beyond the pencil mark at each end, and by 10 a.m., 2 ft beyond. Shores were immediately placed under this panel. It was through investigation of this rapidly sagging panel that the most serious damage to the structure was brought to light. Around the column at each corner of the panel the mastic flooring was bulged and hollow. By taking up this covering the workmen disclosed concrete shattered as if with a sledge hammer over the entire area of the negative steel—a distance of 5 ft 6 in. from the center of the column along both center lines. Underneath the shattered concrete the steel was in a deplorable condition both from corrosion, the force that had disintegrated the comparatively thin overlying layer of concrete, and from electrolysis.

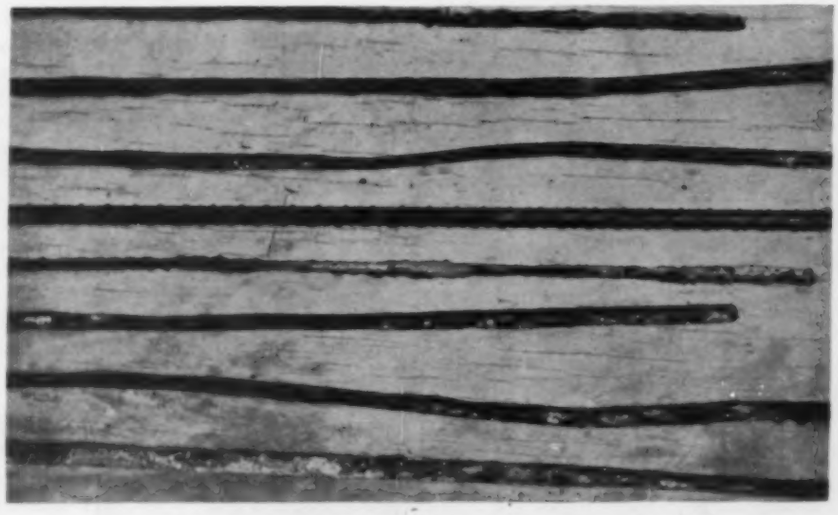
A piece of the floor concrete spalled off by the force of this corrosion is shown in a photograph. In it the rust adhering to the concrete from the bars may be seen. The surrounding dark area is the oxidized liquid product of the electrolysis. Some of the bars taken from the corroded negative steel of a panel are shown in another photograph. Tension tests on these bars gave breaking strengths from 2,200 to 3,500 lb. Of the 750 lin ft of negative steel in the area around a support, on an average of not more than 40 lin ft was fit for use again, and this was all in short lengths. Fully three-quarters of the negative steel areas on the first floor were affected in this way and to an equal degree.

The electric conduits in the first-floor slab were generally placed in or near a construction joint, and these too were badly damaged. In many places the walls of the conduit were eaten through, and the conduit was filled with salt water. The insulation on the wires was so soaked that probably there was a leakage of current, although this was not checked with instruments.

METHOD OF REPAIR

The problem of repairing the building and making it fit for continued use as a hide warehouse logically divided itself into three parts: first, to get rid of all affected steel and sources of electrolysis; second, to replace the old steel with new and to add the proper fireproofing; and third, to make it as difficult as possible for any salt water to get to the steel in the future. To determine the details of each part of the problem and to help in arriving at a rational solution to each, experiments were made on two basement columns. The method developed there was used throughout the repair work with only very minor changes.

As the first step in the repair of a column, shores were placed around the outside of the drop panel. These were designed to carry the first-floor live and dead loads, leaving the loads of the second floor and roof to the concrete core of the column. Since the building had been designed for four stories, although it had only been built to two, this procedure was safe. It gave a stress in compression in the core not to exceed 450 lb per sq in. The next step was to chip the concrete from around the vertical bars, removing the spirals as the chipping progressed. The steel was cleaned of concrete to a minimum distance of 2 in. beyond any sign of rust to make sure that all damage was uncovered. The steel was then brushed with a wire brush, chipped with a small cold chisel on all sides, washed with limewater, and carefully inspected. It was necessary to chip the steel very thoroughly because the pits resulting from the electrolysis were extremely hard to detect. They were often under the rust or the original mill scale of the bar. Washing with limewater after all the cleaning was finished dissolved and neutralized any material remaining in the pits.



SAMPLES OF STEEL REMOVED FROM SLAB AROUND COLUMN
A New Bar Included for Comparison

After this thorough cleansing, new steel was added to make up the deficiency in the reinforcement, and a new spiral was wrapped around the vertical bars. At this time four more shores were added close to the column capital, and the jacks were screwed up to take as much

of the load from the column core as possible. Then a 1:4 mix of gunite was shot on to the column to bring it out to its original size or to give the correct amount of fireproofing to the steel. At the floor line a cove of 8-in. radius was formed when the gunite was shot. A view of the completed column shows this cove.

Repairs to the wall consisted in chipping the concrete from the steel 3 in. beyond all damage, cutting out all steel affected either by corrosion or electrolysis, and welding in new steel. All welds were made by the electric-arc method, using a 2-in. lap weld with a $^1/_2$-in. bead on $^1/_2$-in. square bars. The concrete fireproofing was then replaced with gunite and the wall waterproofed. A core of 8-in. radius similar to that at the base of the columns was formed where all the walls and pilasters met the floor line.

In repairing the slabs, the area on which the work was to be done was adequately shored from below. Then the concrete was cleaned from around the steel with air hammers. In the case of the expansion cracks and construction joints, the slab was cut through from the top so that all damaged concrete was removed at the same time. First the new steel taking the place of that damaged was arc-welded in place, and then the old steel was cut out by the same method. The concrete was replaced by gunite of 1:3 mix. In the negative steel panel around each column, the concrete surrounding the steel was removed with air hammers 3 in. beyond all signs of damage, and the affected steel removed. New steel was then arc-welded in place and the concrete replaced with gunite. In these panels the gunite was sloped away from the column to the corners of the drop panel, and a drain put in each corner so that if at any time there was a leakage around the column the water could not stand on the slab and cause possible future damage. Water dripping into the basement through these drains would also be an indication that there was a leak in need of attention.

It was required that the waterproofing for the concrete should keep the water out and at the same time be capable of resisting severe blows from hand trucks and small wagons used in the warehouse. After some experimenting a combination of asphalt enamel and softened asphalt enamel reinforced with burlap was developed on the job that filled all requirements perfectly. First the concrete was primed either with an emulsified asphalt or a cut-back asphalt primer—they were equally effective. Then one coat of asphalt enamel that had been softened with gasoline was put on hot and in it was embedded a sheet of burlap. Over the top a heavy coat of straight asphalt enamel was applied hot. This produced a thick coating of asphalt hard and tough enough to withstand without damage the hardest blow that could be given by a man with a carpenter's hammer.

PIECE OF CONCRETE FLOOR SLAB SPALLED OFF BY CORRODING NEGATIVE STEEL

Waterproofing of the walls was done similarly except that no burlap was embedded in the soft asphalt coat. In repairing the slab, it was necessary to take up the asphalt mastic, and it was essential that this be put back with absolutely tight joints. The old mastic from the floor was remelted with one part in three of new mineral-filled asphalt of local manufacture and with sufficient dry, hot sand to give the proper consistency. Particular care was taken to secure solid bond at the joints, and special tools were developed on the job for this purpose. Test cuts through the joints after several weeks of service showed the bond to be so perfect, and the old and new mastic so homogeneous, that the exact line of joining could not be detected. A special joint of asphalt enamel and mastic was made at the base of each column to ensure against leakage.

INTERIOR OF FIRST FLOOR AFTER REPAIRS WERE COMPLETE
Hide Curing Brine Drained Away from Columns

To secure some information as to how closely practice approximated theory, some test blocks were prepared of steel surrounded with gunite and submerged in the salt solution. Pieces of uncleaned steel as they came from the walls, pieces of cleaned steel, such as those left in the concrete, and pieces of new steel were cast in blocks with a 1-in. gunite covering. These blocks were then immersed in the solution without waterproofing of any kind. At the end of three months samples were taken out, broken open, and examined. No action of any kind had taken place on any of the steel bars. Of course this is too short a time for a conclusive test, yet it does show that the gunite will give good protection by itself. After being in place three months, the new steel in a section of the basement wall was tested with a high resistance voltmeter as before. The difference in potential was only 0.01 v., which is believed to be negligible under present conditions.

This building was further tested by the earthquake of March 10, 1933. Although hollow tile partitions in the building suffered and there was much broken glass in the steel sash, there was no evidence of any structural weakness in the building. Repeated careful examinations have revealed no cracks in the columns, slabs, or waterproofing.

The Circular Boundary of Delaware

Arc with Its Center at New Castle, Del., Dates from Days of Charles II of England

By Lynn Perry

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CONSIDERABLE current interest attaches to the circular boundary of Delaware because it has a close bearing on the recent boundary controversy between that state and New Jersey. The final opinion in this case was delivered by Justice Cardozo of the U. S. Supreme Court on February 5, 1934. An account of the various events leading up to the creation of this unique boundary line in 1681 and of the several surveys that have modified it, is of interest not only to the engineer but also to the attorney and the historian. It was original with, and was established at the request of James II of England while he was Duke of York and while the grant of Pennsylvania to William Penn was being considered.

ENORMOUS areas of land in the Colonies were lavishly granted to royal favorites in the first half of the seventeenth century by James I and Charles I without a great deal of attention being paid to such details as boundary lines. The first grant for the Colony of Virginia by James I, April 10, 1606, included the land between the 34th and 45th parallels of north latitude. This comprised most of North America from about Columbia, S.C., to Halifax, Nova Scotia. The charter granted in 1609 increased the privileges, and that of 1612 added the Bermudas, but the territory thus given to the London and Bristol companies was materially reduced in 1620 by the same king, when he granted the charter for the Plymouth Colony, comprising the land between the 40th and 48th parallels, extending from the Atlantic to the Pacific oceans. This included the territory from Philadelphia to near the center of Newfoundland. In June 1632, Charles I, the son of James I, granted to Cecil Calvert, Second Baron of Baltimore, a charter to Maryland between the 38th and 40th parallels of north latitude, and extending from the Atlantic Ocean and Delaware Bay to the headwaters of the Potomac River. This lay entirely within the then recongized boundaries of Virginia, but her charter had been annulled in 1624.

During the revolution and the interregnum that followed, when Cromwell and his "Roundheads" were in power, the English were too much occupied with home affairs to devote much attention to America. But with the accession of Charles II in 1660, a wave of joy and good will swept over the country and the authorities again turned their attention to the Colonies. Four years later Charles II granted to his brother, the Duke of York (afterwards King James II), all the territory between the west bank of the Connecticut and the east bank of the Delaware.

WILLIAM PENN'S CHARTER

When William Penn asked for a grant of land in America, a great deal of consideration was given to American charter boundaries. Those whose rights might in any way be affected were consulted and all objections were given consideration. The Duke of York, later James II, was the friend and patron of Penn because of his father, Sir William Penn, who won great renown in the Battle of Lowestoft. Action was held in abeyance for several months. Curiously enough, the conflicts over the boundaries of Penn's grant were more bitter and more protracted than those of any other English colony. The charter, as it was finally issued in 1681, granted Penn the land between the 40th and 43d deg of north latitude, and extending from the Delaware River west 5 deg of longitude.

Penn was anxious to get as near the seaboard as he could, and in the conferences prior to the granting of the charter, the Duke of York was asked how much territory he thought he should retain in the vicinity of New Castle. The Duke agreed to relinquish his claim, re-granted to him in 1674, to the unsettled parts of the upper Delaware and suggested that a boundary 20 miles north of New Castle would be suitable. After further consultation between Penn and the Duke, a distance of 12 miles was agreed upon and the charter was issued. By it the Duke retained his colonies on the lower Delaware although they were not mentioned in either of his grants from the King, these extending only to the eastern banks of the Delaware. So Pennsylvania's southern boundary was described as "following a circle drawne at twelve miles distance from New Castle, Northward and Westward (from the river Delaware) until the beginning of the fortieth degree of Northern Latitude, and thence by a streight Line Westward to the Limit of Longitude."

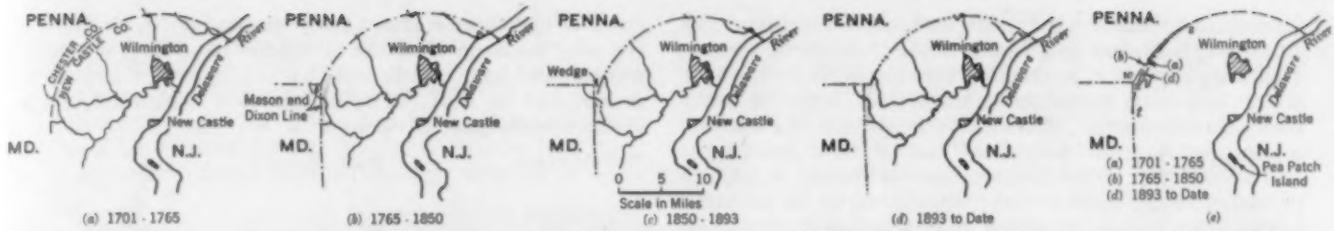
Penn seems to have become dissatisfied with his title, and on August 21, 1682, he obtained from the Duke a quitclaim deed, relinquishing to Penn any claim that the Duke might have to the Province of Pennsylvania. Three days later the Duke gave Penn two deeds of feoffment for Delaware. The first conveyed to Penn the town of New Castle and a 12-mile circle around it. The second conveyed all the lands south of that circle as far as Cape Henlopen. The title of the Duke was

Mason and Dixon Marker (Rounded) and Graham Stone

Placed at Point *A* in Fig. 2, Where the Western Boundary of Maryland, Going North, Becomes Tangent to the Arc

Stone Placed by Graham at Northeast Corner of Maryland

Placed in 1850 Over Part of Foundation of Original Mason and Dixon Corner, Point *C* in Fig. 2



Arc Run by Isaac Taylor and Thomas Pierson | Maryland Boundary as Marked by Mason and Dixon | The Boundary as Determined by Lt. Col. J. D. Graham | As Finally Determined by Pennsylvania–Delaware Joint Commission | Wedge Gained by Delaware and Strip Lost Along the Arc in 1893

FIG. 1. EVOLUTION OF THE 12-MILE ARC FORMING THE PENNSYLVANIA–DELAWARE BOUNDARY

shadowy and that of Penn could have been no better. True, the Duke was exerting governmental jurisdiction over Delaware from New York, but he had no grant to the lands, and Penn's agents experienced great difficulty in their attempts to collect rents in Delaware. It is worthy of note, also, that the King (Charles II) never confirmed Penn's claim to the lower Delaware, nor did the Duke of York do so during his reign as James II. The deed to New Castle and a 12-mile circle around it contained a covenant for further assurance any time within seven years. At the request of Penn, the Duke promptly took steps to rectify his title. Letters patent under the Great Seal of England were issued in March 1682, granting New Castle, together with the land and subaqueous soil within a circle of 12 miles radius around it, to the Duke of York. Delivery must have been made promptly, for on October 28, 1682, we find Penn at New Castle, where formal livery of seisin of the lands and waters within the 12-mile radius was made. He gave the colony a liberal government, incorporating the Delaware settlements with Pennsylvania proper, and allowing them representation in the Provincial Assembly. However, the Delawareans soon became dissatisfied with Penn's administration and requested a separate assembly and administrative officers. This was granted during Penn's second and last visit to Pennsylvania in 1701. Delaware was thereafter called the "Three Lower Counties."

THE FIRST SURVEY

When the Three Lower Counties were separated from Pennsylvania, it became necessary to establish a boundary dividing New Castle from Chester County. This survey was authorized by Penn in August 1701, when Isaac Taylor of Chester County and Thomas Pierson of New Castle County were appointed surveyors to run and mark the boundary between those counties—a circular arc of 12-mile radius with the town of New Castle as its center, and extending from the Delaware River westward two-thirds of a semicircle (120 deg). This arc is shown in Fig. 1(*a*). They began their survey on September 26 at the "End of the Horse Dike next to ye Town of New Castle" and measured "thence Due North the Distance of twelve miles...." The survey was completed October 4 and certified nine days later. The field notes of Taylor and Pierson as well as those of Mason and Dixon are in the possession of the Pennsylvania Historical Society.

Running this radius and arc in nine days, including a Sunday, through forest and across streams with a compass and Gunter's chain was no small accomplishment. The lineal distance measured was over 37 miles, an average of over $4^1/_2$ miles per day. The surveyors ran a "due north" line 12 miles, which took them to a peculiar bend of the Brandywine Creek shown on Figs. 1 and 2. They made a considerable error in both angle and distance. This point, which they assumed to be 12 miles "due north" of New Castle, is about 0.9 mile too far to the west and about 2,000 ft too far from New Castle. If they had measured along the surface of the ground, their distance would have been less than 12 miles. So it is probable that they measured horizontally, and still their error indicates a chain about 2.1 ft too long. This appears ridiculously large but is not improbable with the equipment and personnel available in America in 1701. If the links were worn only one-fourth through, the chain would be approximately 2.65 ft too long.

Their error in westing might be explained by their not taking into account the magnetic declination. But there is a record of an observation for magnetic declination in Philadelphia in 1701, and it was 8°30′ W. This would have carried them a half mile further to the west, and besides, they probably knew their declination. A more probable explanation would be that they were using a poor needle and that this was unduly affected by local attraction, particularly noticeable in the valley of the Brandywine.

They laid off a line at right angles to their supposed true radius. After computing the distance corresponding to the chord of one degree of a circle of 12 miles radius as 67 perches, they laid off half of this on the east perpendicular to the radius and half on the west. Their value of 67 perches is sufficiently accurate (67.018 would have been exact) but considering how much their radius was in error, it is probable that the chords as actually laid out were somewhat over 69 perches in length. Then they moved to the eastern end of the half-chord, laid off one degree to the right by compass, measured another chord, and continued thus for 43 chords, which brought them to the house then occupied by Daniel Lamplaugh on the banks of the Delaware. The western end of the curve was laid out similarly, ending "at a stream, a branch of the Christiana Creek." This stream was probably Persimmon Run, and the point was probably in Maryland near Iron Hill. As near as could be determined at the time of the next survey, in 1892, the curve had a radius of about 13 miles. It was marked by blazed trees and served as a Colonial and state boundary for many years.

THE MASON AND DIXON SURVEY

Mason and Dixon located the northeast corner of Maryland, *C* in Fig. 2, in 1765, and both Delaware and Pennsylvania assumed this point to be the junction of the three states. The lines then appeared as in Fig. 1(*b*). The marked points of the former survey in that vicinity were disappearing and the actual line was becoming increasingly difficult to find on the ground. Little heed was given to the line, and very few deeds or grants to land specified it as a boundary. However, for a mile or two east of the Brandywine, property lines

were made to conform to the line and still continue to do so.

Although Mason and Dixon made their survey before Delaware became a separate state, a part of their work, which had some influence on these lines, must be introduced at this point. The western boundary of Delaware separating it from Maryland has a true bearing of N.3°36′06″ W. This line was run northward, according to instructions, until it intersected a circle of 12 miles radius, the center of which was the Court House in New Castle, and thence due north to the northwest corner of Maryland. Coming from the south with a bearing a little west of north, the line became tangent to the circle at *A*, Fig. 2. If it had been produced due north from the tangent point, it would have cut off a thin segment of the circle, leaving a part of Maryland less than 12 miles from New Castle. So the line was established from the tangent point to the west and north around a part of the circumference of the 12-mile circle until it reached a point due north of the tangent point (*B*, Fig. 2). The boundary was then run due north to the northeast corner of Maryland. This cut out of Maryland a small segment of a circle containing about 12 acres.

They marked the tangent point, the center ordinate, and the south end of the due north line with dark granite monuments without any identification marks. The best known of these stones, that at the tangent point, is in a meadow between the Pennsylvania Railroad and the highway (Route 2) leading from Wilmington to Baltimore, about 2⅞ miles west of Newark. Mason and Dixon fell into some errors in placing these three stones; they are 108 ft too far from the belfry of the New Castle Court House, and the chord is about 300 ft too long.

THE GRAHAM SURVEY

In the course of time, the boundary marker at the northeast corner of Maryland disappeared. The uncertainty of the officials of all three states as to the exact location of the boundary led to the appointment of a joint commission in 1849. Legislation authorizing this commission on the part of the three states leaves little doubt that the unmarked corner was considered to be common to all three. This commission secured the services of Lt. Col. J. D. Graham, of the U. S. Corps of Topographical Engineers, an unusually well-qualified engineer of broad experience and profound technical knowledge. He reestablished the northeast corner of Maryland (*C*, in Fig. 2) by producing the north and east lines of that state, from monuments then in existence, to an intersection. This corner was marked by a large granite post still in excellent condition. According to his report, subsequently published by each of the three states, the position of this stone is within a few inches of the original Mason and Dixon stone because he discovered part of the foundation of the old stone while excavating for the foundation for this one.

His work was completed, but he proceeded to lead his commission to make a decision concerning which it had not the slightest authority. Recognizing the poor condition of the three stones on the arc above referred to, he reviewed Mason and Dixon's computations, found their errors, and proceeded to remark the arc bulging out into Maryland and to place more permanent monuments at the southern end, middle point, and northern end of the arc. The last two he placed in new locations where his computations indicated that they should be. The last of the three he declared to be the true common point of the three states, and so marked it (*B*, in Fig. 2). He also declared that the northern boundary of Delaware should have a radius of 12 miles instead of about 13, as laid out in 1701. The commission concurred in his decision and signed his report.

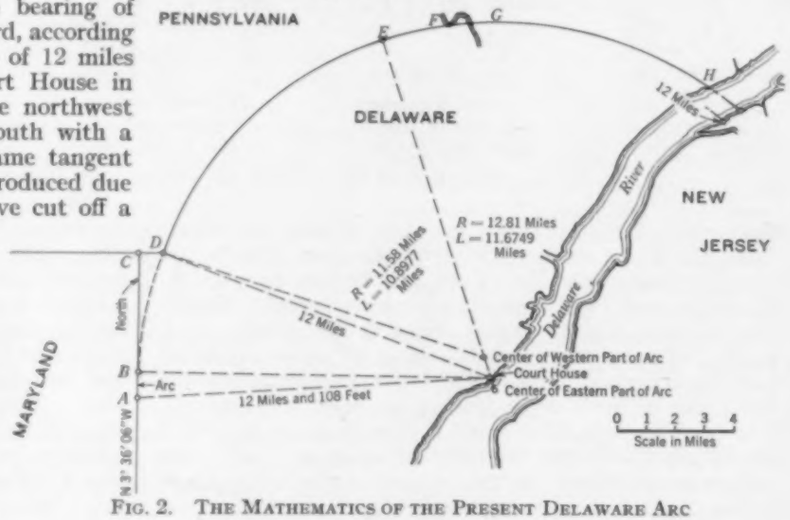


FIG. 2. THE MATHEMATICS OF THE PRESENT DELAWARE ARC

This placed the boundary as in Fig. 1(*c*). Maryland was perfectly satisfied with the action of the commission, not claiming any territory east of the old Mason and Dixon line. Pennsylvania was pleased, standing to gain "the wedge" jutting southward between Maryland and Delaware as well as a circular strip about a mile wide south of the line laid out by Taylor and Pierson in 1701. Delaware, standing to lose territory over which she had exerted jurisdiction since early Colonial days, was the only loser. Why her commissioner should have agreed to such action, or why Colonel Graham should have advised the commission to abandon ancient boundaries and lay out new ones is beyond explanation. It certainly was not granted any such authority by the legislation which created the commission, and it is doubtful whether even the legislatures would have had such authority without a plebiscite.

NEW BOUNDARY ACCEPTED ONLY ON PAPER

No subsequent acts of ratification seem to have been passed by any of the state assemblies. Pennsylvania took steps looking forward to a resurvey and final settlement in Acts of Assembly passed in 1850 and in 1869. The action of the commission seems to have been accepted on paper but otherwise ignored. It was "accepted on paper," because maps made between 1850 and 1893 show Pennsylvania as jutting down in a wedge between Maryland and Delaware; it was "otherwise ignored," because the citizens living in the wedge (*BCD* in Fig. 2) continued to vote, serve on juries, and pay taxes in Delaware. Pennsylvania had never claimed any territory south of the Mason and Dixon Line, nor within the Delaware arc as laid out in 1701. In fairness to the Pennsylvania authorities, it should be stated that no attempt seems to have been made to annex the territory.

Delaware maintained almost normal jurisdiction over the wedge, and at least one member of her legislature, William Smith, came from that territory. Obviously if the U. S. Courts had been called upon to hand down a decision in the case, the action of the commission would not have been sustained, and the former status, as shown in Fig. 1(*b*) would have resulted. Even

Colonel Graham's computations were not without error, as shown by the survey of 1892.

ANOTHER COMMISSION APPOINTED

After the lapse of some forty years, most of the animosities created by the Graham Survey had died, at least sufficiently to warrant another attempt to settle

MONUMENT SET IN THE SURVEY OF 1893
Point of Beginning of Present Curve, *D* in Fig. 2

MONUMENT ERECTED BY STATE OF PENNSYLVANIA
Where a Main Road Intersects the Arc

the boundary dispute. A joint commission composed of two boards, from Delaware and Pennsylvania, was authorized and appointed in 1889. Benjamin H. Smith of Philadelphia was named "surveyor on the part of Pennsylvania," and Daniel Farra "surveyor on the part of Delaware," each to act also as secretary to his respective board. These two boards seem to have acted separately, with their technical and clerical appointees, searching records and identifying various points of the arc on the ground. From time to time, the two boards met together, sitting as a commission and agreeing on procedure, policy, and other points that would naturally require resolutions of agreement and accord.

After considerable time and a very thorough and diligent search, they found only four, or possibly five, tolerably authentic points marked by Taylor and Pierson between the northeast corner of Maryland and the Delaware River nearly two centuries before. These were:

1. The remains of the old house near the Delaware River, occupied by Daniel Lamplaugh in 1701, at which Taylor and Pierson ended the eastern part of their arc (Fig. 2, *H*).
2. The boundaries of some farms lying between the Concord Pike and the Brandywine Creek (Fig. 2, *F* to *G*) the original grants for which called for the circular arc as their boundary.
3. A peculiar bend in the Brandywine Creek (Fig. 2, *F*), where it comes down to the Delaware line, turns back into Pennsylvania, and again turns southeast into Delaware. This was described by Taylor and Pierson very clearly as the northern terminus of their 12-mile radius.
4. A large hickory stump (Fig. 2, *E*), which marked the junction of the boundary of Kennett and Pennsburg townships, Chester County, and the State of Delaware. This was mentioned as a small hickory in 1713 in a deed by George Harlan to his son, James, for a part of the "manor of Staneing" granted by William Penn to his daughter Laetitia in 1701.

West of this point not a single place was found, the authenticity of which could be established under the rigid rules adopted by the commission. They found a number of places, pointed out by the inhabitants along the line, that were being used as the state boundary but they refused to accept usage or tradition as sufficiently reliable.

During the survey of 1892 and after the commission had decided to accept the intersection of Colonel Graham's 12-mile circle with an eastward prolongation of the Maryland-Pennsylvania boundary as the beginning point of the arc, another point on or near the survey of 1701 was uncovered. A stone which marked the corner of a farm in Mill Creek Hundred dated back to about 1830. At that time, land of an intestate decedent lay across the state line, and the courts of both states were interested. A surveyor was employed to establish the state boundary across the land. This was done, the stone was placed, and jurisdiction over its respective part was assumed by the court of each state. This surveyor's notes were not available but he undoubtedly performed a conscientous task for the court. The stone was certainly within a few feet of the original survey and could well have been accorded some consideration.

Since the commission refused to award this point the weight it undoubtedly deserved, the curve was brought from *x* to *y* in Fig. 1(*e*), a distance of some 2,000 ft. It ceded to Pennsylvania the triangle *xyz*, infuriating the population and landowners involved in the transfer and regenerating the old animosities caused by the survey of 1850. This delayed the final establishment of the new line another three decades. In perspective, it appears as though the Delaware commissioners were so anxious to clear their state's title to the wedge that they were willing to make concessions along the arc. At the time it was not known what kind of curve would fit the points agreed upon as a boundary, and it is doubtful that the Pennsylvania commissioners had any desire to take from Delaware any territory over which it had been exercising jurisdiction for nearly two centuries. The commission had decided to find and remark the old line of 1701 as nearly as possible, and if the Delaware commission had requested a reconsideration of the decision to start the arc at *x*, it is probable that the request would have been granted. Nevertheless, the final result was that Delaware lost her title to the triangle *xyz* as shown in Fig. 1(*e*), and cleared her title to the wedge *twy*.

After deciding to have the curve laid out in this way (Fig. 2, *DEFGH*) and to locate suitable boundary markers, the commission secured the services of W. C. Hodgkins, of the U. S. Coast and Geodetic Survey, under whose direction the survey was made.

Field work was begun in 1892 and finished the following year. After an accurate triangulation survey had been made and these five points accurately located, it was seen that the arc of no circle would fit them. A circle would, however, fit *EFGH*, and another circle of shorter radius would fit *DE*. This made a compound curve with the point of compound curvature at *E*, where the boundary between Kennett and Pennsburg townships has its corner on the original arc. It was decided upon, laid out, and the stones were set. The state lines then became as in Fig. 1(*d*). Although Delaware ex-

tends along this curve to the low-water mark on the eastern side of the Delaware, this survey did not cross the river. On Fig. 1(*e*) are placed the curves shown in (*a*), (*b*), and (*d*). The curve in (*c*) is of 12-mile radius, as suggested by Colonel Graham, instead of about 13 miles, as in the other surveys.

THE BOUNDARY STONES

To mark this boundary in as permanent a way as was feasible, the initial stone was erected on the prolongation of the northern boundary of Maryland, and 4,169 ft east of the northeast corner of that state as established by the survey of 1850. It is dark Brandywine granite. The mile and half-mile stones along the curve are very durable gray gneiss, about one foot square, with a flat top, standing about 2 ft above the ground. They are set in concrete foundations and are probably as durable and permanent as is practicable.

MILESTONE NO. 13, PLACED IN 1893 ON THE PRESENT ARC

The half-mile stones are marked with "1/2" on the west side. The mile stones have "P" on the northern side and "D" on the southern; "1893" on the eastern side and the number of miles from the initial stone on the western. The initial stone bears the names of the commissioners representing Pennsylvania and Delaware on the north and south sides, respectively. On the east side is "1893" and on the west side notation bearing witness to the fact that the field work was performed by the U. S. Coast and Geodetic Survey. Reinforced concrete posts such as the obelisk-like monument shown in a photograph have been erected by the Pennsylvania Department of Highways along the main roads leading out of the state.

THE CURVE BETWEEN MARYLAND AND DELAWARE

The curve, *AB* in Fig. 2, projecting into Maryland has a radius of 12 miles and 108 ft, and its center is at the New Castle Courthouse belfry. It was marked by Colonel Graham in 1850 at the beginning, center, and terminus of the arc.

The part of the curve *DE*, in Fig. 2, has a length of 10.8977 miles and a radius of 11.58 miles. Its center is some three-fourths of a mile northwest of the New Castle Courthouse belfry. The arc *EH* has a length of 11.6749 miles and a radius of 12.81 miles, and its center is nearly half a mile southeast of the same point. From *D*, which is exactly 12 miles from the Court House belfry, the distance from the courthouse gradually increases to *E*, where it is 12 miles plus 1,877 ft. Thence the increase is more gradual to *H*, where the distance is 12 miles plus 3,137 ft. As the curvature of the earth had to be taken into consideration in computing and laying out this curve, the work was not as simple as that of laying out a compound railroad curve, where the principles of plane trigonometry govern. The survey of 1892 was ratified by the Pennsylvania Assembly in June 1897, by the Delaware Legislature in March 1921, and by Congress in June 1921.

From the beginning of its statehood, Delaware has been engaged in a boundary dispute with New Jersey. The decision of February 5, 1934, terminates the suit entered in the U. S. Supreme Court praying for a determination of the boundary between the two states in the Delaware River and Bay. After a thorough investigation by a Special Master, the following decree was entered:

"Within the 12-mile circle, the river and the subaqueous soil thereof up to the low-water mark on the easterly or New Jersey side will be adjudged to the State of Delaware, subject to the Compact of 1905.

"Below the 12-mile circle, the true boundary between the complainant and the defendant will be adjudged to be the middle of the main ship channel in the Delaware River and Bay."

Many of the more important reasons for this decision are indicated in the preceding pages. Considering the area within the circle first, these will be outlined briefly. The Duke of York and Albany was in possession of this territory by right of conquest and exerted a de facto authority over it from 1664 until he awarded it to William Penn in 1682. To support his title, he obtained a grant from the king. In treating with the New Jersey authorities, Penn was careful to maintain his right to the river bed.

In 1813 the Delaware Legislature ceded Pea Patch Island, shown in Fig. 1(*e*), to the United States for the erection of Fort Delaware. This island lay in the Delaware River within the circle, far to the east of the main ship channel. New Jersey protested, but after years of litigation, decision was rendered in favor of the United States.

In 1877, New Jersey started proceedings in the U. S. Supreme Court to establish her claim to the disputed territory in the river bed. The case was discontinued without prejudice in 1907. The compact of 1905 provides for the "enjoyment of riparian rights, for concurrent jurisdiction in respect to civil and criminal process, and for the concurrent rights of fishery." It states: "Nothing herein contained shall affect the territorial limits, rights, or jurisdiction of either state of, in, or over the Delaware River, or the ownership of the subaqueous soil thereof. . . ."

As regards the river and bay below the circle, the main reasons for the decision are as follows. Before 1783, when the Colonies became independent, the title to the subaqueous soil of the Delaware River and Bay was in the Crown. No grant to this had been issued to either New Jersey or Delaware. For this reason the court applied the doctrine of the "thalweg." The "thalweg," or "downway," is the track taken by boats in their course down the stream, which is that of the strongest current. The center of the channel of a stream has been considered as a boundary for over a century, and the court has applied this doctrine in many such cases.

It would now appear that Delaware's boundary disputes have been finally settled. All that remains is a survey and a few boundary markers. Lt. J. H. Brittain of the U. S. Coast and Geodetic Survey has been engaged to perform this service and the work is now in progress.

In the preparation of this article, use has been made of the decision of the U. S. Supreme Court dated February 5, 1934, not yet published, and of original manuscripts relating to the Taylor and Pierson survey and Mason and Dixon survey in the archives of the Pennsylvania Historical Survey. Information has also been obtained from various publications.

Foundation Experiences with Clay in Texas

Special Treatments Found Efficacious to Provide for Settling and Swelling Action in Plastic Soils

By Willard E. Simpson

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IN this article, prepared from a paper delivered before the spring meeting of the Texas Section in May 1934, Mr. Simpson presents some very practical advice to those who would build on plastic clay soils. In them more than in any other foundation material does the presence or absence of water play an important part in determining stability or suitability as a support for heavy loads. To prevent settlement of a building after it has been constructed, it is usually impractical to carry the excavation to such a depth that the natural superimposed soil load is equal to the building load to be applied. Settlement must therefore be expected even in undisturbed soils, the permissible amount being measured by the differences in the settlement of various piers and their probable effect on the building. It may be safer to permit a tall rigid structure of large load concentration to settle 5 in. than to allow a frame residence to settle one inch. Clay soils act somewhat like a sponge, contracting and cracking as they dry out and swelling with great force as they take up moisture. Mr. Simpson maintains that, with proper construction on clay soil in its natural state, settlement may be reduced to a safe amount and destructive heaving largely prevented.

Gulf Building, Houston, Tex., Erected During 1928

IT was not so long ago that even engineers looked upon a foundation as just so much concrete and steel, or so many piles driven into the ground to certain depths. Even today there are innumerable patented types of foundations and all kinds of intricate systems for arranging the steel and concrete. And with it all, very little thought is given to that on which the concrete and steel must themselves rest. (It may not be generally known that soil can be in a state of stress from 50 to 100 ft below the point of application of the foundation load; that this stress is not uniformly distributed under the footings; that a layer of soft mud or quicksand 20 ft below the footing level may seriously increase settlement; or that an adjacent structure can have a marked effect on the settlement of a new building.) Yet these factors and many others materially affect the settlement of any structure placed on soil.

Scientific research into the action of soils has proved beyond doubt that the conditions in question are of great importance. Research has also shown that soils have one strength or characteristic when in their original condition and a very different strength or bearing value after they have been disturbed.

It should be realized then that the foundation of a structure is the soil itself and that its classification and action, and its characteristics under load, are the determining factors in the building of a safe foundation. The science of soil mechanics has received a great deal of attention in the last six years, notably since Charles Terzaghi, M. Am. Soc. C.E., prepared his treatise on the "Science of Foundations," published in Transactions, Vol. 93 (1929). Since then, laboratory methods have been developed whereby soils can be properly sampled, their action under stress studied, and their classification determined. However, the more laboratories study the science, the more involved it becomes. The mechanics of soils is really a science of great magnitude and scope and worthy of the efforts of the most able engineering minds.

Reference will be made here only to the several types of plastic clays, since well-compacted sandy clays or sandy soils cause little trouble when more or less confined. Sand, of course, is the very best foundation material and is generally an unyielding one. Excessively sandy clays also form stable, unyielding foundations and are not easily disturbed. In Texas it has been found that most foundation troubles are caused by plastic clays. There is no difficulty in using rock, hardpan, or beds of gravel as foundations, if they have the proper thickness.

One of the first of the perplexing phenomena of soils found in the testing laboratory is that when a soil is disturbed by man or by nature its character is entirely changed. In its undisturbed state a soil is stable and capable of resisting well-defined stresses without undue strain, but after it has been disturbed just the opposite is true. This fact leads to two immediate conclusions: first, in order to secure a stable, reliable foundation, the natural structure of the soil or clay should not be disturbed; and second, in seeking a foundation for footings soil that has had its natural structure disturbed in any way should be avoided.

An entirely suitable foundation soil may be ruined during excavation by allowing ground water seeping from above the foundation level to run down on it. Also, men walking on the surface churn a 6 or 8-in. layer into a mush that is entirely unsuitable for a foundation and will start to be compressed as soon as any weight is placed on it. When the stratum of clay desired for the foundation is reached, extreme care should be taken to keep all outside moisture off the surface. A desirable practice is to require, immediately before the concrete is poured on the soil, that the top 2 in. which have been disturbed by the drying-out effect of the air be peeled off to a level, firm, moist stratum.

One of the greatest natural factors tending to disturb soils is the evaporation of capillary water, which is constantly rising toward the surface. Natural clays of

either marine or terrace plane deposit are filled with such water and when this is driven out, no human being can restore the clay to its original structure or give it back its original ability to carry loads. At the surface of the ground, the heat from the sun's rays evaporates the capillary water as fast as it rises. This evaporation is

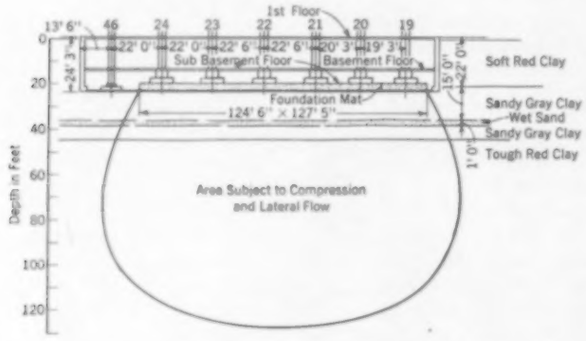

FIG. 1. FOUNDATION AND LOG OF FOUNDATION MATERIAL Gulf Building, Houston, Tex.

complete down to a certain point and then continues with gradually diminishing effect until at a certain depth below the surface, the original amount of water is present. In the vicinity of San Antonio the effect of the sun's heat extends to a considerable depth, on an average to 15 or 20 ft below the surface. Below this point the clay deposits are undisturbed but above it they are very dry and porous, and of a crumbly, coarse nature, filled with air instead of capillary water.

A soil of this sort is easily compressed under load. Not only is it doubtful as to what load, if any, such a soil will carry before excessive compression occurs, but also these dry coarse soils are unstable because they are thirsty for moisture. Immediately after a structure is placed on them, thereby shielding them from the sun's rays, capillary water again rises and softens them. Also, moisture from the surface can run into them, softening them until they are unsuitable for foundations.

After a suitable foundation clay has been found the greatest problem still remains. All soils or clays compress and flow under load, and they continue to do so, to an extent depending on the intensity of the load applied, their plastic state, and their depth. Marine deposits, that is, clays deposited entirely under water, are not as dense or as good for load bearing as terrace plane deposits, which have been alternately deposited under water and dried.

Say that a certain soil, 20 ft below the surface, has carried a load due to the earth above, of 2,300 lb per sq ft. Under this pressure it has reached a point of equilibrium as far as further consolidation or settlement is concerned; that is, the resistance of the water that can be driven out under pressure is just balanced by the weight of the soil above. If on this soil at this depth a structure is placed which increases this pressure, further consolidation will occur and settlement of the structure will result and continue until the soil again reaches a point of equilibrium. It will take longer for the second point of equilibrium to be reached when the additional load is high than when it is comparatively low. If it is excessive for the particular soil, or if there is a sandy layer below the bottom of the footings, which will readily absorb the water pressed out of the clay, the soil may never reach a state of equilibrium and settlement may continue indefinitely.

The real foundation problem is not what bearing capacity a soil has, but how much weight can be placed on it above that which it has been previously carrying due to the surcharge load of the earth above the foundation level. In some structures the total additional load is the total weight of the structure and its live loads. In others, such as buildings with deep basements, it is this weight minus the weight of the soil excavated.

This latter case brings up an interesting factor, which is often overlooked. This is that in buildings having partial basements the footings immediately under the basement should be given a higher unit pressure than the other footings. This difference in pressure should equal the pressure that had been exerted by the additional soil excavated from the basement. (It is often noted, especially in the case of schools, that there is a general settlement of the entire building with the exception of the part under which there is a basement.) A natural conclusion is that the basement walls and floors create a higher resistance to settlement in this part of the building, when as a matter of fact the footings under the basement doubtless were calculated for the same intensity of pressure as the remaining footings without allowance for the reduced weight on the soil due to basement excavation.

It is conceivable that occasionally, where only a soft foundation soil is available for a rather heavy building, the reduction in pressure due to basement excavation to extra depths may permit such soil to be used as a foundation. Enough soil can be taken out for the basement so that no additional pressure is placed on the soil beyond that which it originally carried.

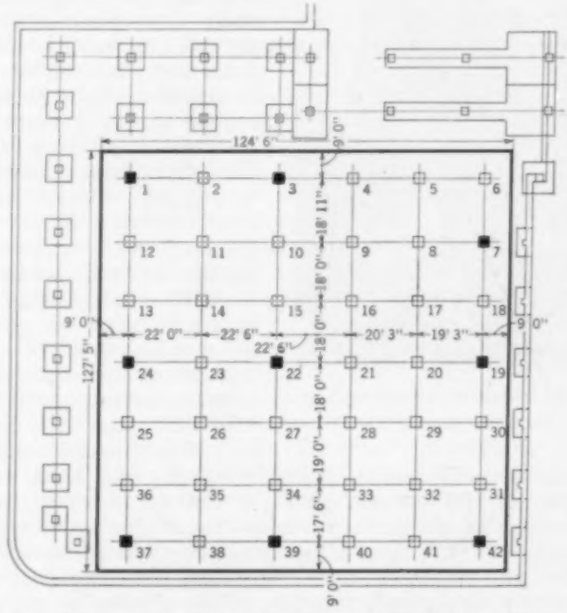

FIG. 2. FOUNDATION PLAN SHOWING, IN BLACK, COLUMNS ON WHICH SETTLEMENT READINGS WERE TAKEN

After the foundation soil has been selected and the weights to be added are known, the next point to determine is how much total settlement can be allowed for the structure in question. It has been found that there is a relation between the total settlement of a foundation and the differential settlement, that is, the difference between the maximum and the minimum settlement. A very large total settlement will generally result in an excessive differential settlement. In some types of buildings this differential settlement cannot be even $\frac{1}{4}$ in. For instance if two adjacent footings of a resi-

dence have a differential settlement of from 1/4 to 1/2 in., destructive cracks will occur. In a two-story school building a larger differential settlement is permissible, which means that a larger total settlement can take place. In a very tall structural steel and reinforced concrete frame office building, sometimes as much as 2 in. of differential settlement can be allowed, which means that 4 or 5 in. of total settlement can take place. In some types of bridges very little differential settlement can be tolerated; therefore, the total compression of the soil must be kept low.

When a load is placed on the clay, above that under which it was deposited, settlement immediately begins and continues at a rapid rate through a certain initial period and then gradually diminishes. After another period, settlement practically ceases. The additional weight that can be placed on a clay must be determined from the total settlement which the structure will stand and the length of time required for that settlement to take place.

If engineers would keep accurate time-settlement curves of each structure founded on clay, it would be comparatively easy to determine the pressure to be allowed on a similar soil in other cases. However, there are many conditions that affect the time-settlement curve besides the character of the clay; among them are the total thickness of the stratum and the presence of layers of other materials below it, such as clays of different softness, quicksand, or silt. These will make it necessary to explore to a considerable depth below the foundation proper. Laboratory methods have been perfected whereby time-settlement curves can be made from different soil samples. The samples are taken by means of an apparatus designed by S. J. Buchanan, Jun. Am. Soc. C.E. By means of this apparatus undisturbed soil samples can be taken from the clays and placed in a testing machine designed by Mr. Terzaghi. Both of these machines are discussed in the Progress Report of the Society's Special Committee on Earths and Foundations, published in the May 1933 issue of PROCEEDINGS.

One of the first discoveries I made in studying foundation problems was that a certain soil would stand a very high pressure under a heavy, stiff structure without injury to it but would not safely carry a residence. In San Antonio there is a certain clay soil at a depth of 25 ft below the surface on which I would not dare add more than 1,500 lb per sq ft under a residence, or 2,500 lb per sq ft under a school. Yet at the present time this very soil is successfully carrying the Plaza Hotel with an added soil pressure of 6,000 lb per sq ft. The total settlement in this building has undoubtedly been several times that of a residence. Yet the added pressure on this soil under a residence must be kept very low in order to keep down the differential settlement.

Although the time-settlement curves made in the laboratory are extremely useful to the designer of a foundation, those of the actual settlements of a building are the most valuable. The making of such curves for actual buildings is very difficult and extends over long periods of time. It is rather expensive and is often discouraged by the owners because they dislike to have the public know that their buildings are settling. Such curves for one of the most important buildings in Texas, the Gulf Building in Houston, may be of interest. This is an office building 35 stories high. Up to the time it was built the common method of preparing the foundations for tall buildings was to drive concrete piles into the clay subsoil on which the footings rested. Feeling that the non-compressible, impervious clay soil into which the piles would be driven, would not be in any way consolidated by the piles but that rather they would disturb it to such an extent as to soften it and possibly increase the settlement, or at best, merely transfer the pressure to the clay at the points of the piles, I advised the omission of piles. Also, this procedure would result in a saving in the cost of the building of more than $60,000.

After much careful study it was decided to place a raft or mat foundation under the entire building, of such area that the total dead load would be kept at 5,000 lb per sq ft and the dead and live loads and wind pressure together would not exceed 6,600 lb per sq ft. It will be noted from the soil section (Fig. 1) that to a depth of about 20 ft there was a rather soft red clay. The next 25 ft was a gray clay with a stratum of sand about 1 ft thick. This gray clay was filled with numerous small pebbles and was of medium plasticity. Below the gray clay there was a fairly tough red clay extending down more than 100 ft below the surface. From rather crude preliminary time-settlement tests made on the latter it was roughly determined that a total pressure of 6,000 lb per sq ft would produce a total settlement of this building of about 3 in. over a period of 5 or 6 years. Since it was a distinct departure from the usual type of foundation used in Houston up to that time, it was decided to keep a record of the settlements of the building. This record has been kept and will be continued for several years in the future.

The mat foundation (Fig. 2) supports 42 columns of the tall tower part of the building, the weight on the columns averaging about 2,300,000 lb each. Nine columns were selected on which to keep a record of settlement. Immediately on top of four column footings resting on this mat round-headed bolts were placed in the concrete to serve as elevation points. Points on the first floor were selected on the remaining five columns

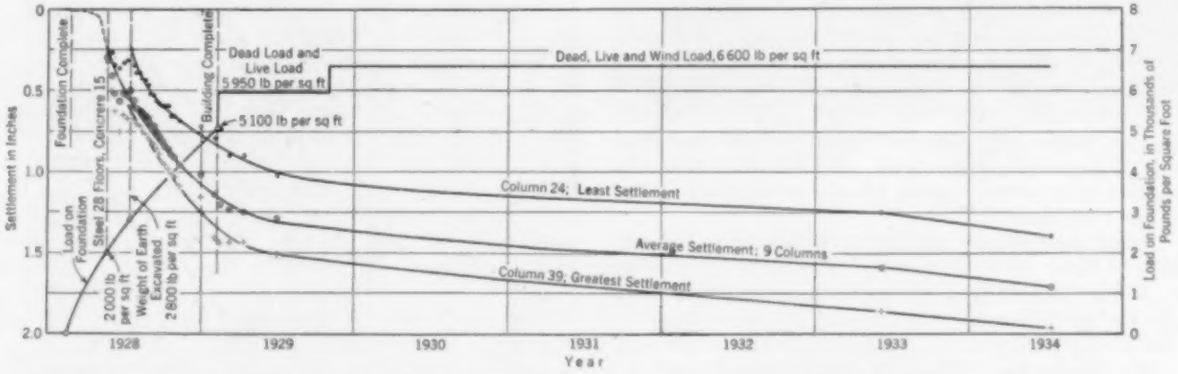

FIG. 3. TIME-SETTLEMENT CURVES WHICH INDICATE AN INCREASE IN FOUNDATION LOAD DURING CONSTRUCTION

and these points later transferred to permanent points in the masonry.

Pouring of the foundation concrete was started in the latter part of February 1928. Settlement readings were started the latter part of May of that year, when the structural steel frame was up about 25 stories, representing a pressure on the soil of 2,000 lb per sq ft. At this time the entire mat showed a small settlement. At the depth of the footings, 26 ft, the undisturbed soil pressure had been 2,800 lb per sq ft.

In Fig. 3 is given a curve of increasing weights of the building, designated as "load on foundation" during construction and until completion in February 1929. At completion, the dead load on the foundation amounted to 5,100 lb per sq ft. On this figure also are shown the additional pressure lines, the first representing the combined dead and live load of 5,950 lb per sq ft, or a total dead, live, and wind load of 6,600 lb per sq ft. The middle, time-settlement curve represents the average settlement of the nine columns. The upper curve represents that column showing the least settlement during the period, that is, Col. 24, and the lower curve that column showing the greatest settlement, Col. 39.

A smooth curve has been put through the irregular line of total settlements to make it easier to visualize the trend. It will be noted that at the beginning while weight was being added to the foundation, the settlements were pronounced and the curves rather steep, but that after completion of the building, the curve began to flatten out. Six months after completion the curves had become extremely flat. To date, six years after the building was completed, the total average settlement is between $1^1/_2$ and $1^3/_4$ in., while the total differential settlement, that is, the difference between the upper and the lower curve, is about $^5/_8$ in.

It was anticipated that the total settlement would be between 3 and 4 in. Of course, this determination was made from rather crude and unreliable time-settlement tests made on the soil in place. When the building was designed I did not know of the possibility of having time-settlement curves made in a laboratory; otherwise this procedure surely would have been followed. However, it would seem that the original estimate of settlement probably will not be exceeded in 10 years.

In this building it will be noted that, considering the dead load only, the original soil produced a pressure of 2,800 lb per sq ft and the building that of 5,100 lb per sq ft. In other words, a load of 2,300 lb per sq ft was added to the soil, and it is this additional load that is causing settlement. The additional pressure was considerably reduced in this case by excavating the basement to a greater depth than was at first anticipated. When the foundation depth was determined a space of about 6 ft remained between the top of the mat and the established basement floor level. This 6 ft was not backfilled with earth as might have been done, but was left as an additional basement. By this means the soil was relieved to 700 lb per sq ft of pressure, which assisted in reducing the total settlement. No special design was made to endeavor to stiffen the building to resist differential settlement. It was felt that the nature of the structure, with all its partitions, would supply the necessary stiffness.

To avoid disturbing the natural structure of the foundation clay, a 3-in. reinforced concrete working slab was laid immediately after the clay was excavated to final depth. Thus the foundation soil was protected while the work of placing the steel in the mat progressed.

Another perplexing problem encountered in the design of foundations, particularly for buildings, but also for highways, is the swelling tendency of certain soils found throughout Texas, more especially immediately around San Antonio. If a basement slab, a first-floor slab, or a sidewalk is placed directly on any soil in this locality which has stood exposed for a number of years, it is practically certain that the original level of the slab will change. Moisture entering the soil causes it to swell with great force. At one time I attempted to measure the intensity of the swelling force of some Texas soils and found that a certain yellow clay often encountered in this territory, which contains concretions of lime or chalk and has been disturbed, will exert a pressure in swelling of over 8,000 lb per sq ft. Maximum swelling does not occur in the surface soil but in that from 3 to 6 ft below the surface. Basement slabs of buildings placed at this level, have been known to swell as much as 18 in. where no provision was made for drainage. We are now making studies of methods to prevent swelling in a particular building in San Antonio where the basement slab, about 6 ft below the surface of the original ground, has already bulged 12 or 14 in. in the last four years, completely wrecking all the partitions in the basement, forcing them up against the first floor, and causing damage there also.

There are two sources of the moisture which causes this swelling. Probably the most common is capillary water from some underground reservoir forcing its way up from below. Before construction of the building the level of this water was kept down by evaporation. After construction, the building offers shade and the water again rises, impregnates the soil immediately under the slabs, and causes swelling. Another source of water is leaky mains or pipes, which invariably will be found under such buildings. The swelling soil breaks the pipes and the leaks so caused in turn aggravate the swelling.

After trying for several years to find a method of preventing this swelling I have almost arrived at a solution. When capillary water gets into the soil the only way to get it out is through air ventilation, or heat combined with air ventilation. Considerable success has resulted from a series of drain-tile trenches placed under slabs. The trenches are generally dug about 4 ft deep, and an open tile drain is placed in the bottom. The function of the drain is to keep the reservoir of underground water at least 4 ft below the slab. The trench is then filled, not with small gravel, which would form an obstruction to the flow of air, but with very large stones about 6 in. in diameter and all practically the same size, so that there will be a free flow of air in the trench. These trenches are spaced not over 12 ft apart. At intervals along them open grates are placed in the floor in order that the circulation of air may be maintained. Of course, the tile drains are connected to sumps, and pumps keep the water below the 4-ft level. Striking results have been obtained by installing such drains in buildings where the slabs have already started to bulge with a great deal of force. Immediately after the drains were placed, the swelling stopped. In some instances where the floor was in a display room at the street level, tunnels were dug under the slab to accommodate the drainage system. Immediately after the tunnels were dug, and even before the drain tiles or large stones were placed, the lifting of the slab ceased.

Such swelling of the soil is noted in many concrete highways, where it produces the so-called high joints. It will be of interest to see just what method of solving this problem will be developed by highway engineers. A way has been found for buildings, but the same method probably would not be practical for highways. The same force is the cause of both failures.

A Comparative Analysis of Plate Girders

Use of T-sections for Girder Flanges Has Numerous Advantages

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ESSENTIALLY, the usual built-up steel grider consists of a web plate to which flange angles are riveted. This conventional type is here compared with girders built up of web plates riveted to wide-flanged I- or H-beams which have been split longitudinally to form T-sections, the halves forming the two flanges. The use of split-beam T-sections for girder flanges has a number of advantages, among them being a larger web-rivet capacity; an appreciable reduction in the weight of the finished girder; and, where heavy shearing stresses must be handled, the possibility of using much shallower girders because of the ability to connect a larger amount of flange material to the web. Because the use of T-sections for flanges of girders is a basic improvement in girder construction, the authors have applied for and received patents on it.

ADVANCES in shop practice in recent years have made the production of split-beam T-sections not only practical but easy. Huge pieces have been produced by this method, the largest to date being over 92 ft long and weighing 7 tons. In smaller work one company has found it worth while to roll beams in certain sizes for the express purpose of splitting to obtain T-sections, since this method has proved more advantageous than rolling T-sections as such.

New fields in structural development, riveted and welded, are being opened up by the introduction of this shape. The split-beam T-section has been used as a chord section for roof trusses, as a wind-bracing connection, and as an ordinary beam framing connection. It is also used to advantage as the element of a plate-girder flange. In this construction its function is identical with that for which the original section was designed. To withstand flexural stresses efficiently the greater part of its material is placed near the extreme fiber. Whether used as a beam, or split and used as part of a girder, this allocation of metal is particularly efficient.

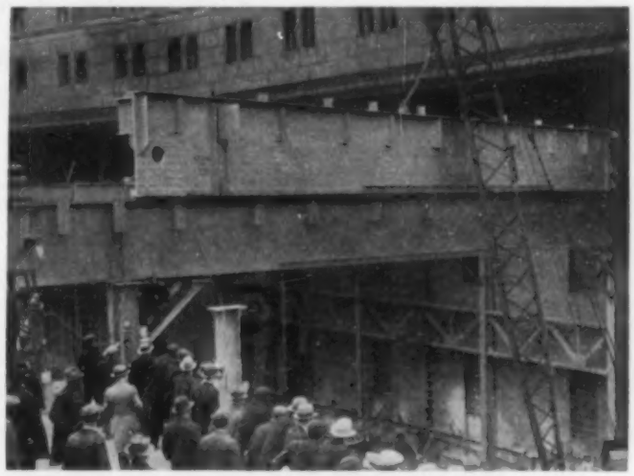
Since over 1,000 tons of T-flange plate girders have been used to date, an investigation of their properties is in order. It will be helpful to analyze the split-beam T-section as a flange element in girder construction and to compare girders of this style with the conventional kind. The subject matter divides itself into three main parts—flexural efficiency of the flange, horizontal shear capacity of the connection between flange and web, and types of web plates. These will be treated in order.

FLEXURAL EFFICIENCY OF THE FLANGE

In Table I girders are classified roughly as light, medium, and heavy. In each subdivision two flanges of about the same magnitude are shown, the first the conventional type and the second the T-type. For the light girder flange, Type II, the T-beam forms the entire flange. For the medium weight girder, Type IV, the T-beam is combined with angles and side plates to form the flange. The side plates serve the double function of permitting the use of web plates of any required thickness, independent of the size of the fillet of the "T," and of improving the adaptability of the flange to variations in section along its length. No rivets are needed through the crosshead of the "T," since all the pieces of the flange are connected by the web rivets. In the heavy girder with T-flange, Type VI, plates are inserted between the horizontal legs of the angles and the crosshead of the "T." These serve the same functions as the side plates in Type IV but are more efficiently placed.

A correct rating of flanges requires a standard of comparison. Here the standard of comparison is taken to be the "ideal" flange shown in the last column of Table I. It consists of a unit area concentrated at the extreme fiber of each flange at the center line of the girder, varying to zero at the supports according to the ordinates of a parabola. The efficiency of actual flanges is less than that of the ideal for three reasons: first, holes for rivets must be deducted; second, the center of gravity of the practical flange is not at the extreme fiber, thus making the effective depth less than the over-all depth; and third the variation in section along the length of the girder is roughly stepped and only approximates the theoretical bending moment curve.

In Table I the six flanges shown are rated by weight in comparison with the ideal flange. Thus the increase in weight caused by the deduction for the holes of the practical flange as compared to the ideal flange is the gross flange area divided by the net flange area. This is represented by A, the weight coefficient for holes. The increased weight of the practical flange as compared with the ideal, caused by the poorer location of the center of gravity, is the ratio of the over-all depth to the effective depth of the girder. In Table I an over-all depth of 72 in. was used for all girders. This ratio is expressed by B, the weight coefficient for depth. To obtain C, the weight coefficient for variation in section, the material was stopped off according



ERECTING VIADUCT OVER RAILROAD TRACKS IN PITTSBURGH
14 Girders 6 Ft 8 In. Deep, Up to 92 Ft 6 In. Long Used. Note clear Flanges and Absence of Intermediate Angle Stiffeners

to a parabolic bending-moment diagram, in which due allowance was made for additional length of pieces to develop their strength.

An average weight per foot was thus obtained, which, divided by the weight per foot at the center line, gives the weight of the practical flange as compared with the ideal. The product of these three weight coefficients, *A*, *B*, and *C*, is the combined weight coefficient, which is a true index of the efficiency of the flange. The smaller the coefficient the lighter or more efficient is the section.

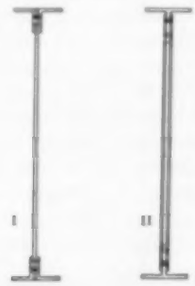
In the light class it is seen that the advantage of the T-flange comes from the smaller deduction for holes. In the medium class the T-flange lags behind the angle flange in effective depth (*B*) and variation in section (*C*), but more than makes this up by the smaller deduction for holes (*A*). The combined coefficient is about 5 per cent less for the T-flange. Actual examples have shown that this saving in tonnage is real, varying in individual cases from zero to about 7 per cent. In the heavy class the conventional girder is less efficient because of the location of the side plates and because of its poorer variation of section. In it the side plates must run the full length, whereas in the T-type (VI) the only flange material that extends all the way through to the end of the girder is the "T" itself.

It is apparent that this analysis is predicated upon the "flange area" method of designing girders. Accurate computations by the moment of inertia method have shown that the relationships given are approximately correct. The analysis so far has not considered web plates, which for light girders change the picture considerably. In Type II practical considerations specify a $^5/_{16}$ or $^3/_8$-in. thickness for each web as the minimum, thus requiring $^5/_8$ or $^3/_4$ in. as the minimum total web thickness. Usually a thin web is all that is needed where the flange is so light, so that the excess web material of Type II may be considerable. It may therefore be stated that in general Type I is more efficient than Type II.

In the medium class, where the girder is subject to a light shear, T-flange Type IV may again waste web material as compared with the conventional Type III. Here the relative merits of the two types must be determined by examining each weight coefficient, since Type III is more efficient in the web, and Type IV is more efficient in the flange material and has a lower shop cost. Also, there is considerably less riveting in Type IV, two lines of rivets being required in each flange instead of six as in Type III. This means of course less handling of pieces and less punching before assembly. There is also less handling after assembly, since all the riveting is done through the web. In cases where a web $^5/_8$ in. or more thick is required, the comparison is clearly in favor of the T-flange, Type IV.

Outside the field of efficiency it may be noted that

TABLE I. STUDY OF FLEXURAL EFFICIENCY AND CAPACITY OF VARIOUS PLATE-GIRDER FLANGE TYPES

		Light		Medium		Heavy		Ideal
Types		 I	II	III	IV	V	VI	Theoretical conception—unit area concentrated at extreme fiber of each flange at center line of span, varying to zero at supports according to ordinates of a parabola
Average Flange	Material	2 L s 6 × 6 × $^7/_8$	18 WF 124 (split)	Cover plates 14 × 2 2 L s 6 × 6 × $^5/_8$	24 WF 160 (split) 2 plates 6 × $^1/_2$ 2 L s 6 × 6 × $^3/_4$	Cover plates 24 × $3^1/_2$ 2 L s 8 × 8 × $1^1/_8$ 2 plates 20 × 1	Cover plates 24 × 3 36 WF 300 (split) 2 plates 8 × $^3/_4$ 2 L s 8 × 8 × $1^1/_8$	
	Gross area	19.5	18.1	47.5	46.3	157.5	161.5	1.0
	Net area	17.7	17.7	40.0	43.2	140.0	143.1	1.0
	A, weight coefficient for holes	1.102	1.023	1.188	1.072	1.125	1.129	1.0
	Over-all depth	72.0	72.0	72.0	72.0	72.0	72.0	72
	Effective depth	68.4	68.2	67.7	66.4	60.5	62.8	72
	B weight coefficient for depth	1.053	1.056	1.064	1.084	1.190	1.146	1.0
	Weight per foot at center line	66.2	62.0	161.3	157.8	535.4	549.4	3.4
	Weight per foot, average section	66.2	62.0	134.4	136.0	419.9	415.9	2.27
	C, weight coefficient for variation in section	1.0	1.0	0.833	0.862	0.784	0.757	0.667
	Weight coefficient = $A \times B \times C$	**1.16**	**1.08**	**1.05**	**1.00**	**1.05**	**0.98**	**0.67**
	Number of rivet lines in each flange	2	2	6	2	8	6	.....
Practical Maximum Flange	Material	2 L s	36 WF 300	Cover plates	36 WF	Same as above	Same as above	
	Limitation	8 × 8 × $1^1/_8$ Maximum angles	(split) Maximum split beam T	20 × 3 2 L s 8 × 8 × $1^1/_8$ Proportion of cover plates to angles	300 (split) 2 plates 8 × $^3/_4$ 2 L s 8 × 8 × $1^1/_8$ Maximum T and maximum angles	Larger flanges are too special for consideration here Proportion of cover plates to angles	Larger flanges are too special for consideration here Grip of flange rivets	....
	Gross area	33.5	44.0	93.5	89.5	157.5	161.5	.....
	Net area	30.9	42.9	81.6	84.2	140.0	143.1	.....

there are certain advantages of the clear flange presented by the T-flange Type IV over the cover-plated Type III. Where construction rests on the flange of the girder, as for instance ties in railroad work or slabs in grillage girders, the rivet heads and stepping of cover plates are objectionable. It has also been found that for exposed work, the point where the rivet head meets the cover plate is usually the place where corrosion starts first and proceeds most rapidly.

In the heavy classification, since the webs will also be heavy, they will have the same aggregate thickness for Types V and VI. The advantage of the T-flange Type VI over the conventional Type V therefore applies to the girder as a whole.

From the lower part of Table I an idea may be gained of the range over which these types can be used. Types I and II are of course limited by the commercially available sizes. In Type III the limitation is the ratio of the cover-plate area to that of the whole flange. The profession is in substantial agreement that too much of the material should not be in the cover plates, but disagrees as to the exact proportion. The flange shown, in which 65 per cent of the area is in the cover plates, is, we believe, about the maximum consistent with good practice.

The T-flange Type IV, limited by the available split-beam T-section and angles, has a maximum net area of 84 sq in. This value is large enough to include well over 90 per cent of all girders fabricated. Although girders have been built with flanges very much larger than any shown in Table I, these cases are very special. Such ingenuity is required in the arrangement of the parts, and often so much of the material must be ineffectively

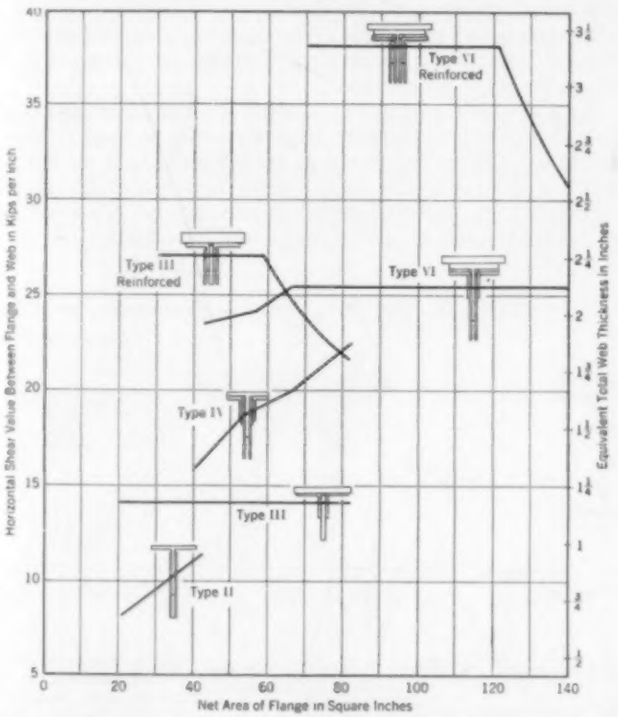
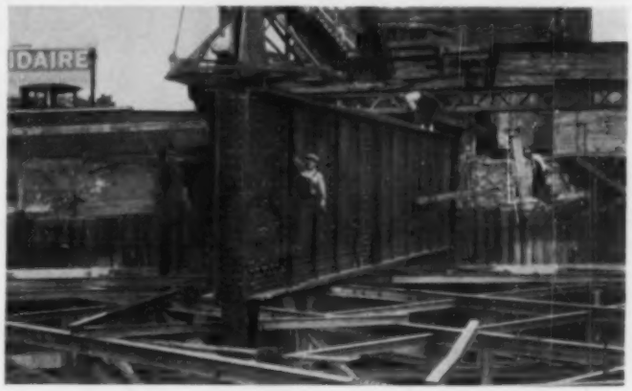


FIG. 1. RESISTANCE TO HORIZONTAL SHEAR OF CONNECTION BETWEEN FLANGE AND WEB

placed, far from the extreme fiber, that in this generalization it is considered unwise to include examples.

In many instances the critical point of a girder design is not the value of the flange in tension or compression, but its connection to the web. Where a large load is to be supported near one end of a girder, the problem is one of shear rather than moment. In this case the necessity often encountered of holding the depth of the girder to a minimum greatly increases the difficulties.

It is simple to pile up cover plates until the required flange area is reached, but it is obviously absurd to have the flange so large that there is not sufficient strength in the horizontal shear connection to the web to utilize its



DOUBLE GIRDER IN U. S. POST OFFICE, PITTSBURGH, PA.
T-beams Made by Splitting a 36-In. Wide-Flange Girder, Weighing 300 Lb per Ft from the Central Element of Each Flange. Girder Designed to Carry Railroad Tracks on Both Sides and a Ten-Story Building Column at Center of 50-Ft Span

full value. Here then is usually the critical point in designing shallow girders for large shears. The girder can be made just as shallow as the horizontal shear value of the connection of flange to web will permit. The same basic thought can be expressed differently by saying that the maximum flange for a case of high shear is that which can be riveted to the web for its full value in the length available. For such cases the ultimate capabilities of shear transference between flange and web of the various girder types are compared.

In Table II is shown the maximum horizontal shear value of each of these types in kips per inch of length, and also the equivalent web thickness on the basis of 12 kips per sq in. of shear area. The value is limited in all cases either by the thickness of the angles or T-flange, or by the maximum number of rivets that the construction permits. In order to obtain a value for a line of rivets, 3-in. spacing for 1-in. rivets is taken as a basis. Thus, in Type II the thickness of the stem of the T-beam, $^{15}/_{16}$ in. in the heaviest wide-flanged section rolled (36 WF 300), limits the value of the connection. In Type III, however, the limit is the value of the two lines of web rivets, which are in double shear. In Type III, reinforced by additional web plates, a new condition is encountered. Here the value of the cover rivets (4 lines in single shear) is only half that of the web rivets (2 lines in quadruple shear). If more than 50 per cent of the flange area is in the cover plates, obviously the cover rivets furnish the limiting condition.

The maximum horizontal shear value of the connection is that of the cover riveting increased by the proportion of the stress carried by the angles. It may be expressed as $\frac{RA_F}{A_C}$, in which R is the value of the cover rivets, A_F is the total net flange area, and A_C the net area of the cover plates. This is an approximation which assumes equal intensity of stress over the entire flange. When A_F equals $2\,A_C$, that is, when the area of the cover plates equals the area of the angles, this horizontal shear value equals that of the web rivets, although here this value (28.2 kips per sq in.) is in excess of the shear value of the angles (27 kips per sq in. for two $1^1/_8$-in. angles). For

larger cover plates the horizontal shear capacity decreases rapidly. This accounts for the sudden drop in the curve for Type III, reinforced, on the horizontal shear chart, Fig. 1. In this figure the horizontal shear values are plotted as functions of the net flange areas. For the same reason the T-flange Type VI, reinforced, drops off suddenly at a much higher value. The slope of the lines for Types II and IV (Fig. 1) is caused by the variation in the web thickness of the split "T" according to the sizes selected.

As illustrated in Fig. 1, the T-flange Type IV is considerably better than the conventional Type III for the whole range of the curve, which at its upper end even rises above the curve for Type III reinforced. The curve for the T-flange Type VI is above that for the conventional Type III, reinforced, for the greater part of its range, while that for the T-flange Type VI, reinforced, is above all the others.

As a word of warning, it should be stated that the horizontal shear charts are for the comparison of types, not for design. The rivet value used as a basis—1-in. rivets on 3-in. centers—cannot be realized in practice for any considerable distance because of considerations of connections, stiffeners, rivet stagger, stopping of plates, and innumerable other details. These considerations in reality affect the angle flanges, Types III and III reinforced, which are primarily controlled by rivets, more than the T-flanges, which are usually limited by the thickness of the stem of the "T."

CONSIDERATIONS AFFECTING DESIGN

In terms of the problem confronting designers, the high value of the horizontal shear transference between flange and web in the T-type girder means the possibility of using thicker web plates and shallower girders for cases of high shear. In the more general case where the shear is not such a critical problem, the connection of flange to web can be considered from another angle.

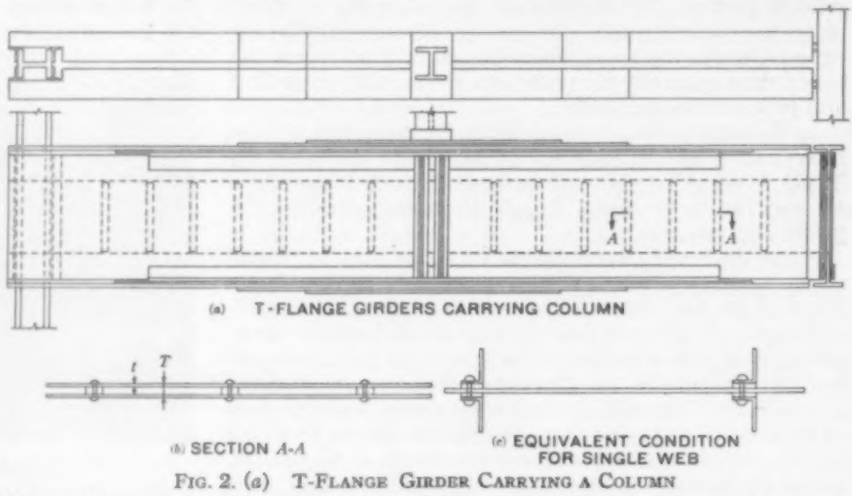


FIG. 2. (*a*) T-FLANGE GIRDER CARRYING A COLUMN (*b*) AND (*c*) STIFFENERS FOR DOUBLE AND SINGLE WEB IN PLATE GIRDERS COMPARED

In Table II the angles, split-beam "T's," and vertical flange plates which are riveted directly to the web plates are cross-hatched in one direction, whereas the cover plates, which are riveted to the angles or "T's," are cross-hatched both ways. The angles and "T's" may be called "once connected" material, since only one system of rivets is required to fasten them to the web, while the cover plates may be designated as "twice connected" material, since they require two systems of rivets for their connection.

In any girder, between a point of zero moment and one of maximum moment, there must be at least enough rivets to convey the stress in each square inch of flange area by shear into the web. Each square inch of "twice connected" material then will require double the riveting of a square inch of "once connected" material. By the

TABLE II. STUDY OF MAXIMUM HORIZONTAL SHEARING CAPACITY Placed in Order of Their

TYPE	II	III	V	IV
CONTROLLING FACTOR	THICKNESS OF T-STEM	WEB RIVETS	WEB RIVETS	THICKNESS OF T-STEM
Explanation (Terminology from American Institute of Steel Construction Specifications)	The largest wide-flanged beam rolled has a web thickness of $^{15}/_{16}$ in. 0.945 in. × 12 kips per sq in. = 11.34 kips per in.	1-in. rivets at 3-in. spacing $\frac{2 \times 21.2 \text{ kips}}{3 \text{ in.}} = 14.1$ kips per in.	Similar to type III. Stress in vertical flange plates, which does not go through angles, slightly increases value	It is always possible to obtain as much material in the angles and plates outside the web plates as in the T-section. Therefore, the maximum value is double that of Type II
MAX. HORIZONTAL SHEAR TRANSFER PER IN.	11.3 kips per in.	14.1 kips per in.	14.1 kips per in. plus	22.6 kips per in.
Maximum Total Web Thickness	$^{15}/_{16}$ in.	$1^{3}/_{16}$ in.	$1^{3}/_{16}$ in. plus	$1^{7}/_{8}$ in.

use of the split-beam T-section the proportion of "once connected" material can always be made greater than in an angle flange and often will form the entire flange. This is translated into a smaller number of flange rivets or frequently their entire elimination.

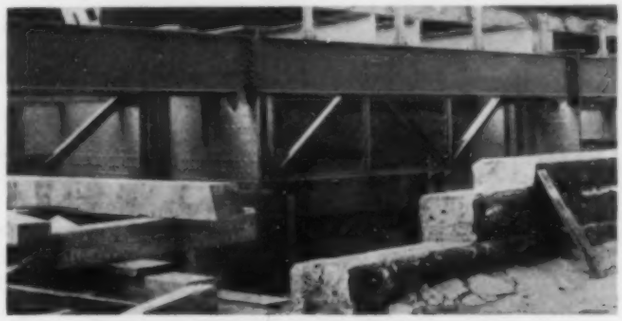
WEB PLATES COMPARED

In comparing the T-flange with conventional types, a basic distinction is at once apparent. The conventional girder has an odd number of web plates and the T-flange girder an even number. As the range from light to heavy girders is scanned, it would seem reasonable that for this feature the advantage would alternate between the types; that is, that it would always be advisable to employ the smaller number of web plates. However, a more exhaustive analysis does not prove this to be always the case.

As is seen in Fig. 2 (*a*), the web plates of T-flange girders are usually braced against each other at intervals by lines of rivets extending through both web plates and through a strip filler placed between them. This is of course unnecessary for shallow girders. As a basis, the old rule of 60 times the thickness, t, of one web plate, for the maximum unsupported distance can be taken as the point below which no bracing against buckling is needed. Where the depth is less than 60 t, no strip fillers are needed. In deeper girders, where they are needed, they should not be farther apart than 60 t.

Now, considering the two plates tied together by lines of rivets and strip fillers, let T be the over-all thickness of the web. If the depth of the girder is not more than 60 T, no additional stiffening of this composite web plate is needed. If the stem of the T-beam is $^3/_4$ in. thick and two $^3/_8$-in. web plates are used, T equals $1^1/_2$ in., and no intermediate angle stiffeners are needed unless the girder is more than 90 in. deep. It is seen that usually the double web does not require angle stiffeners.

For a single web $^5/_8$ in. thick and over, a double web can be substituted without loss of web material. The comparison then becomes one of fittings. It is apparent that strip fillers must be spaced roughly half as far apart as the angle stiffeners for the single web. For one panel of girder the double web needs two fillers, Fig. 2(*b*) and (*c*), as compared with two longer fillers and two still longer angle stiffeners for the single web. Of course it is possible to eliminate the fillers under the stiffeners by crimping, but this requires an additional shop operation. Angle stiffeners requiring machining at the ends to make them fit tightly to the flange angles are expensive pieces to fabricate. Where the space between two web plates is completely sealed from the outside, there is no danger of damage from moisture and subsequent corrosion.



TRUCKING VIADUCT IN PITTSBURGH, PA.
Showing Cantilever End of a Girder in Which Main Flange Element Is a Large Split T-beam

It appears then that in cases where the required thickness of web is $^5/_8$ in. or over, T-flange girders are more economical than the conventional kind; and that where the shear is high, T-flanges, possessing a higher capacity of connection to the web, permit the use of shallower girders.

Illustrations of a few of the installations of T-flange girders are shown in the accompanying photographs. These will give some idea of the practical scope of this type in actual fabrication and erection. It is felt that as engineers become aware of the advantages of such girders in solving ordinary as well as special problems, this type of construction will be found increasingly useful.

BETWEEN FLANGE AND WEB OF VARIOUS PLATE GIRDER TYPES
Ability to Transfer Shear

COVER RIVETS USUALLY CONTROL	WEB RIVETS SOMETIMES CONTROL
1-in. rivets at 3-in. spacing; 4 lines in single shear	2 lines of rivets in quadruple shear = $\frac{2 \times 42.4 \text{ kips}}{3 \text{ in.}}$ = 28.2 kips per in.
$\frac{4 \times 10.6 \text{ kips}}{3 \text{ in.}}$ = 14.1 kips per in. This considers cover plates only. The stress in the angles, which does not go through these rivets, increases the value (Fig. 1)	However, the largest angles are $1^1/_8$ in. thick, equivalent to 2 × $1^1/_8$ in. × 12 kips per sq in. = 27 kips per in.
21 kips per in. plus	27 kips per in.
$1^3/_4$ in. plus	$2^1/_8$ in.

THICKNESS OF T-STEM AND RIVETS IN ANGLES	
Maximum T-stem	
0.945 in. × 12 kips per sq in. = 11.3 kips per in.	
Two lines of rivets in double shear between angles and web plates; $\frac{2 \times 21.2 \text{ kips}}{3 \text{ in.}}$ = 14.1 kips per in.	25.4 kips per in.
25.4 kips per in.	
$2^1/_8$ in.	

THICKNESS OF T-STEM PLUS THICKNESS OF ANGLES	
Maximum T-stem	
0.945 in. × 12 kips per sq in. = 11.3 kips per in.	
Maximum angles are $1^1/_8$ in. thick, equivalent to 2 × $1^1/_8$ in. × 12 kips per sq in. = 27.0 kips per in.	38.3 kips per in.
$3^3/_{16}$ in.	

Shear in Slabs Under Concentrated Loads

Distribution of Stresses Determined Experimentally for a Concentrated Load near the Support

By M. G. Spangler

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WHEN a concentrated load is applied to a floor slab in a structure such as a building or highway bridge, the designer needs to know how much of the width of the slab, called the effective width, is active in carrying the load. Both theoretical and experimental methods have been applied to the problem of determining the distribution of stress in the slab. In this article, presented as a paper before the Structural Section of the Iowa Engineering Society in March 1934, Prof. Spangler reports in detail an addition to the experimental data—his tests on 20 individual slabs at Iowa State College. He also makes a comparison between his data and the previously published analytical results obtained by other investigators, pointing out the differences between the two methods of study and the points in which they do not satisfactorily agree.

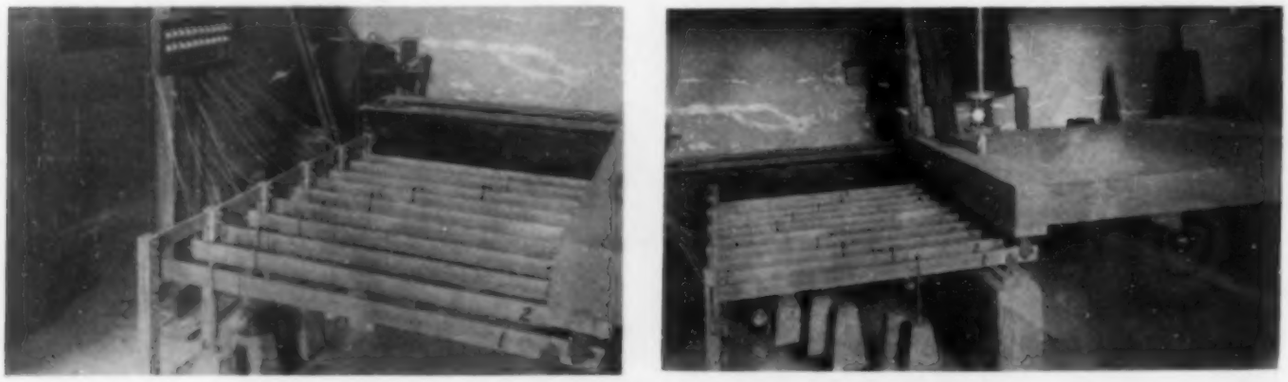
WHEN calculating stresses in a concrete slab due to a concentrated load the designer is immediately confronted with the question of the width of the longitudinal element of the slab that can be safely considered to carry the load. This problem has been recognized since the advent of concrete slabs in the floor systems of bridges and buildings and has become more and more acute, especially in connection with highways, as the weight and wheel concentration of vehicles have increased.

This general problem of stress distribution in slabs has been attacked by several research agencies, notably the U. S. Bureau of Public Roads, the Illinois Engineering Experiment Station, and the Ohio State Highway Department. These three organizations began their studies almost simultaneously, though independently, as early as 1912. The Illinois work was directed by the late W. A. Slater, M. Am. Soc. C.E., and his first results were reported in December 1912 in a paper read before the Ninth Annual Meeting of the American Association of Cement Users. Other papers, by A. T. Goldbeck and C. T. Morris, both Members Am. Soc. C.E., were published soon after. The literature on this subject has since been enriched by other valuable contributions. In *Public Roads* for March 1930, H. M. Westergaard, M. Am. Soc. C.E., presented a very complete analytical solution of the problem. In Bulletin No. 80 of the Ohio Engineering Experiment Station, published in 1933, Professor Morris reviewed the results of the experimental researches and correlated them with Dr. Westergaard's analytical studies.

All the experimental work reported in the literature, however, deals only with the distribution of moment stresses produced by concentrated loads placed near the transverse center line of the slab. There is no record of any previous attempt to study experimentally the distribution of stresses in a slab when the load is placed near one support, which is the position that produces maximum shear in the slab. I began some experiments at Iowa State College on this phase of the problem in 1925, and studies on this subject have since been continued at intervals under my direction. The project was enhanced materially in the summer of 1930, 1931, and 1932 by the work of Guy H. Elbin, M. Am. Soc. C.E., who chose this problem as the subject of his thesis for the degree of master of science, granted him by Iowa State College in 1932. Theoretical analyses of the problem have been made by Dr. D. L. Holl of the Mathematics Department of Iowa State College. Further work of this kind is in prospect.

In all the studies on this subject which are reported here, it has been assumed that the vertical shear on any element of a vertical transverse plane immediately adjacent to the support of a slab is equal to the reaction between the element and the support, and therefore that a measure of the distribution of the reaction will also be a measure of the distribution of the shear. There is evidence that this assumption may not be strictly correct—that the reaction at any point is made up of the vertical shear at that point combined with a vertical force due to certain twisting moments within the slab. Indications are, however, that the difference is small and on the side of safety; that is, the reaction is larger than the shear, so that the assumption will continue to be employed until more specific knowledge is available.

It is customary to express the lateral distribution of



Experimental Slab Set Up for Measuring Reactions by the Lever Method
Left, the Damping Device and Balancing Lights; Right, the Slab and Levers

stress under concentrated loads in terms of an effective width that may be substituted for the width factor in the ordinary formulas for the design of rectangular beams, to make these formulas applicable to slab design. The effective width (Fig. 1) may be defined as the width of an imaginary longitudinal element of a slab over which a concentrated load should be considered to be uniformly distributed in order to produce calculated unit stresses equal to the maximum unit stresses that actually occur in the slab. The use of an effective width for this purpose has been criticized by some commentators, who argue that more direct means of calculating maximum stress should be utilized for slabs, but the practice has become quite generally accepted and will probably continue to be employed.

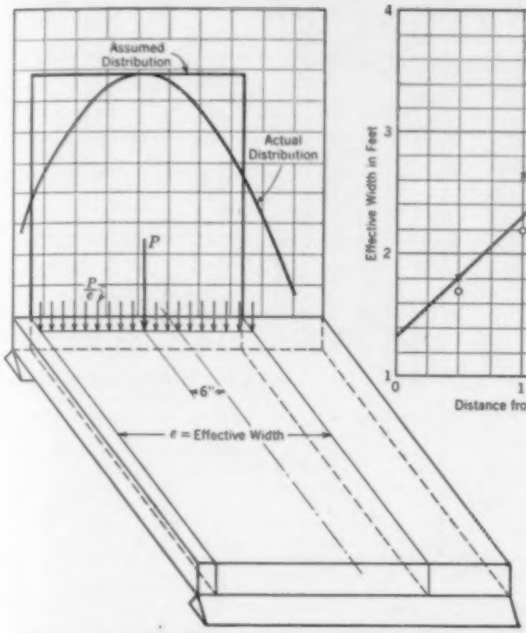


FIG. 1. ILLUSTRATING THE EFFECTIVE WIDTH OVER WHICH A CONCENTRATED LOAD IS ASSUMED TO BE UNIFORMLY DISTRIBUTED

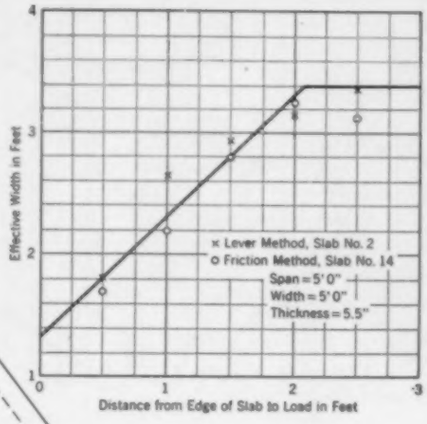

Effective Width in Feet
Distance from Edge of Slab to Load in Feet
× Lever Method, Slab No. 2
○ Friction Method, Slab No. 14
Span = 5'0"
Width = 5'0"
Thickness = 5.5"

FIG. 2. COMPARISON OF EXPERIMENTAL DETERMINATION OF EFFECTIVE WIDTH BY TWO METHODS

Two types of apparatus have been utilized in my experiments, each designed to measure the reaction at one end of imaginary longitudinal elements of the slab equal in width to 0.1 the total slab width. In each of these two methods of measurement entirely different principles of mechanics were employed. The first, referred to as the "lever method," required a series of levers with movable counterweights to balance the reaction on each element. The second, called the "friction method," utilized a series of stainless steel ribbons, which were free to slide between brass surfaces in such a manner that the vertical reaction on each ribbon was proportional to the horizontal pull required to start the ribbon sliding. The friction method was developed as a check on the lever method. It was found that the two methods produced substantially the same results, although judging from the uniformity of the data, the lever method appeared to be the more accurate. A comparison of the effective widths obtained by the two methods for two slabs of identical dimensions is given in Fig. 2. It shows that there is little to choose between the two methods as regards results obtained.

For the lever method of experimental analysis the slabs were built with one end resting freely on a concrete pier and the other on a series of 10 cast-iron bearing blocks, each having a width of $4\frac{1}{2}$ in. in the direction of the span and a length equal to 0.1 the width of the slab transverse to the span. The bearing blocks were entirely independent of one another. Each block transmitted its load through a ball-and-socket joint to a steel saddle, which rested on a pair of hardened knife edges, one on each side and at one end of a cold rolled steel bar about 42 in. long and $\frac{1}{2}$ by 2 in. in cross section. As shown in a photograph, the fulcrum of the lever was 2 in. from the receiving knife edges and consisted of another pair of knife edges resting on an inverted steel saddle, which in turn rested on a concrete pier. A $17\frac{1}{2}$-lb movable counterweight rode the free end of the lever to balance the applied load. Also a 1-lb rider was provided for finer adjustment of the balance.

Early in the development of this apparatus it was realized that some means must be provided to prevent any appreciable vertical movement of the free end of each lever, since excessive vertical movement of one lever might seriously affect the amount of slab reaction being carried by the adjacent levers. Accordingly, a damping device was constructed at the free end of each lever which effectively reduced its vertical swing to an insignificant amount and aided in balancing it.

This damping device consisted of two transverse bars

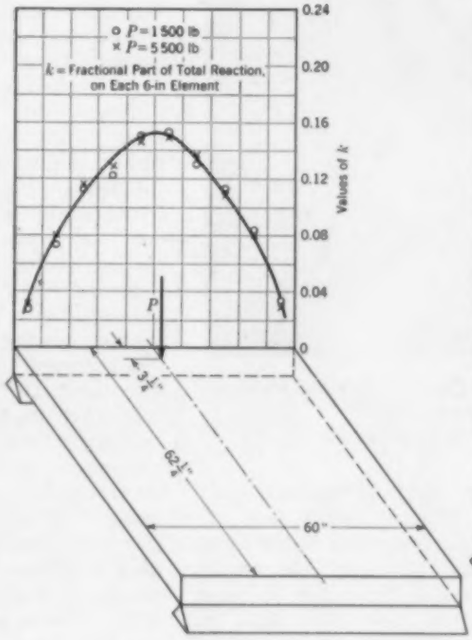


FIG. 3. DISTRIBUTION OF REACTION FOUND TO BE INDEPENDENT OF THE LOAD APPLIED

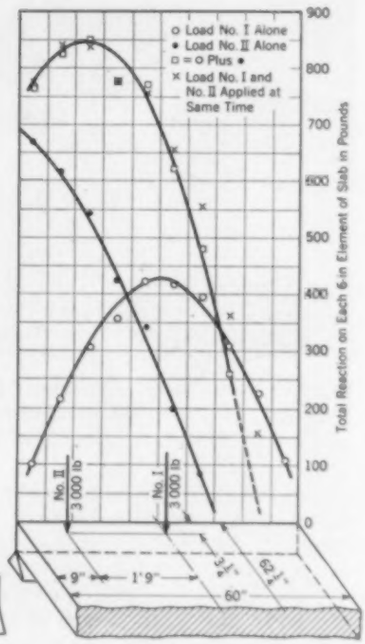

○ Load No. I Alone
• Load No. II Alone
□ = ○ Plus •
× Load No. I and No. II Applied at Same Time
Total Reaction on Each 6-in Element of Slab in Pounds
No. II
3 000 lb
No. I
3 000 lb
9"
1'9"
60"

FIG. 4. SEPARATE REACTIONS ADDED TO SECURE COMBINED REACTION

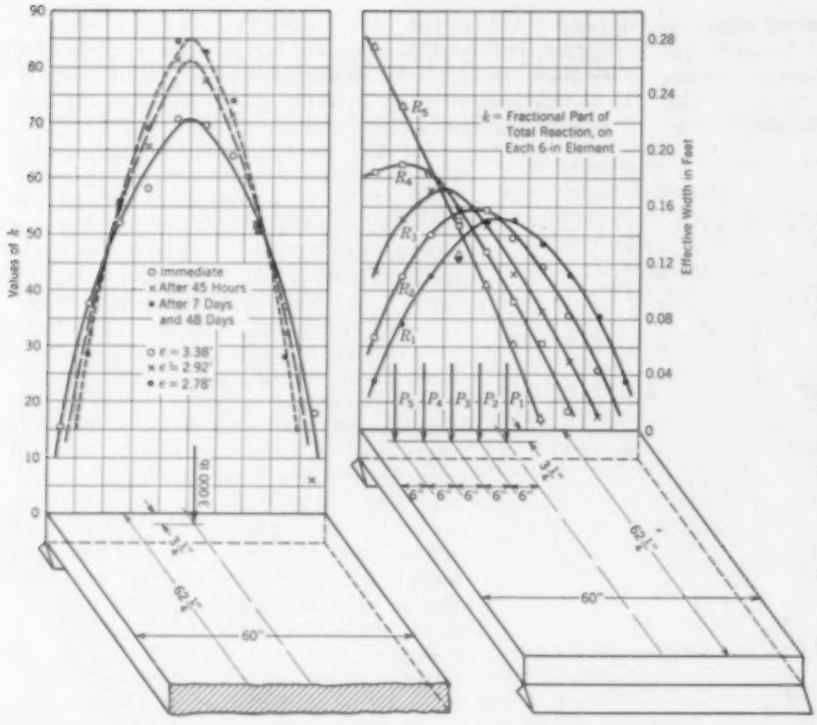


FIG. 5. EFFECTIVE WIDTH CHANGED BY PLASTIC FLOW UNDER A SUSTAINED LOAD

FIG. 6. EFFECTIVE WIDTH AT MINIMUM FOR LOADS NEAR EDGE OF SLAB

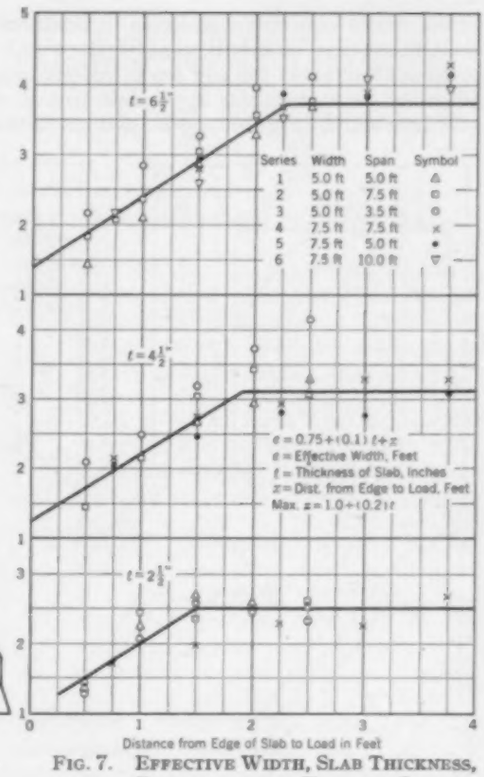
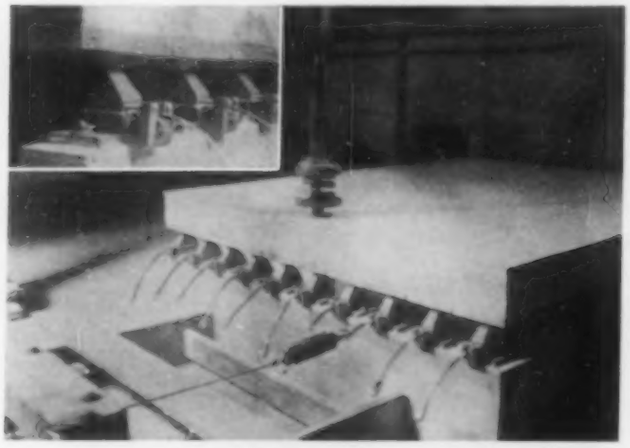

t = 6½"
t = 4½"
t = 2½"
Series Width Span Symbol
1 5.0 ft 5.0 ft
2 5.0 ft 7.5 ft
3 5.0 ft 3.5 ft
4 7.5 ft 7.5 ft
5 7.5 ft 5.0 ft
6 7.5 ft 10.0 ft
e = 0.75+(0.1) t+x
e = Effective Width, Feet
t = Thickness of Slab, Inches
x = Dist. from Edge to Load, Feet
Max. x = 1.0+(0.2)t
Distance from Edge of Slab to Load in Feet

FIG. 7. EFFECTIVE WIDTH, SLAB THICKNESS, AND DISTANCE FROM EDGE OF SLAB

suitably supported from the floor and fitted with adjustable brass screws placed one above and one below each lever. These screws were electrically connected to a bank of small lamps in such a manner that when a lever was in contact with its top screw, the upper unit of a pair of lamps corresponding to that lever would light,



FRICTION METHOD OF MEASURING DISTRIBUTION OF REACTION OF A CONCENTRATED LOAD
In Insert, Close-Up of Friction Block

and vice versa. Then, after the dead load was uniformly distributed over the bearing blocks, which was accomplished by balancing the counterweights at the same distance from the fulcrum on all the levers, the screws were adjusted so that the vertical clearance between the lever and each screw was not more than 0.01 in. When a load was applied to the slab, the various counterweights and riders were moved outward on the levers and adjusted until neither the upper nor the lower lamp for each lever was lit. This indicated that the levers were "floating" between the damping screws and were in balance. The distance from the fulcrum of a lever to its counterweight and rider gave a measure of the reaction on that bearing block.

REACTION BY FRICTION METHOD

In the friction method of measurement, a series of bearing blocks supported the slab exactly as in the lever method. The load from these blocks was transmitted through a ball-and-socket joint to a pair of cast-iron blocks, one above the other. Each of the two surfaces between these blocks was lined with a layer of brass shim stock backed by a thin layer of canvas. A stainless steel ribbon, 0.5 in. wide and 0.008 in. thick, passed between these brass surfaces. The pull required to start the ribbon sliding was proportional to the vertical load on the bearing block depending on the coefficient of friction between the brass surfaces and the steel ribbon. This coefficient was determined in advance of the installation by loading the ribbon assembly in a testing machine.

In order to control the initial vertical reaction on each bearing block, the bottom friction block was ground on the bottom to the shape of a wedge. It rested on a base block ground to the same angle but sloped in the opposite direction. Then by turning an anchored bolt threaded into this friction block, the block could be moved slightly either forward or backward. This horizontal movement increased or decreased the vertical pressure on the ribbon, due to the wedge action, so that it was possible to adjust this pressure to any desired amount. In these tests, as in those by the lever method, the dead load reaction was adjusted equally between the 10 bearing blocks prior to the application of a concentrated load. There is some evidence that this distribution of the slab dead weight is not strictly correct, because of the influence of the twisting moments on the reaction, but this has not been fully demonstrated as yet. When

adjustment for dead load had been completed, the concentrated loads were applied at various points on the slab and the distribution of the reaction measured by pulling the ribbons and noting the pull required to start them in motion. The starting pull was used as the indicator because it was found to be more dependable than the pull required to maintain the motion of the ribbons.

TABLE I. DIMENSIONS OF SLABS TESTED

Slab Number	Series Number	Thickness Inches	Width Feet	Clear Span Feet
Lever Method:				
1	1	4.5	5.0	5.0
2	1	5.5	5.0	5.0
3	1	6.5	5.0	5.0
4	1	3.5	5.0	5.0
5	1	2.5	5.0	5.0
6	2	6.5	5.0	7.5
7	2	5.5	5.0	7.5
8	2	4.5	5.0	7.5
9	2	3.5	5.0	7.5
10	2	2.5	5.0	7.5
11	3	6.5	5.0	3.5
12	3	4.5	5.0	3.5
13	3	2.5	5.0	3.5
Friction Method:				
14	Repeat 1	5.5	5.0	5.0
15	4	6.5	7.5	7.5
16	4	4.5	7.5	7.5
17	4	2.5	7.5	7.5
18	5	6.5	7.5	5.0
19	5	4.5	7.5	5.0
20	6	6.5	7.5	10.0

NOTE: No. 14 was made identical with No. 2 in order to compare the two methods of measurement.

The concentrated loads were applied to the slabs by a screw jack arrangement and were measured by a calibrated coil spring. The diameter of the circular area of contact between the load and the slab was varied from 1 to 4½ in. In addition, one slab was loaded through a wheel equipped with a 4-in. hard-rubber tire. Since it was not possible to note any marked difference in the distribution of reaction due to these different areas of contact, most of the experiments were conducted using as the contact medium a steel cylinder 2 in. in diameter, standing on end.

Twenty slabs have been constructed and tested, ranging in thickness from 2½ to 6½ in.; in span from 3.5 to 10.0 ft; and in width from 5.0 to 7.5 ft. The first 13 of them were tested by the lever method and the rest by the friction method. They were all different in dimension except Nos. 2 and 14, which as stated previously were made identical in order to compare the two methods of measuring the reaction. The data concerning the slabs are given in Table I.

In addition to the main purpose of this study, which was to determine the effective width for shearing stresses in concrete slabs under concentrated loads, several auxiliary measurements and observations were made which are of interest. It was shown that the distribution of the reaction of a slab loaded in this manner is independent of the load, at least within the range of loads that produce unit stresses not greater than ordinary working values (Fig. 3). In these tests, concentrated loads ranging from 1,500 to 5,500 lb were applied to the slabs, but there was no apparent difference in the distribution of the reaction. Also, it was demonstrated that the distribution of the reaction from two loads can be obtained by simply adding the unit reaction at each point due to the loads acting separately (Fig. 4). These facts are in accord with the usual practice in design and are of no particular import, except that it is sometimes valuable to demonstrate anew old accepted principles.

A phenomenon of great interest to designers of concrete structures is the matter of plastic flow of this material. It has been shown by Smith, Fuller, Hatt, and others, that concrete structures, when loaded for a considerable period of time, undergo gradual changes in strain even though the load remains constant. It was interesting in this connection to note the effect of time upon the distribution of the reaction of a slab bearing a concentrated load placed near one support. Such an observation was made on slab No. 2 (Fig. 5). A load of 3,000 lb was placed on the longitudinal center line, close to the support, and held constant for a period of 48 days, the distribution of the reaction being observed at suitable intervals during that period. It was found that the unit reaction opposite the load increased rather rapidly during the first 2 days, after which the rate of increase declined until at the end of the seventh day a constant value of unit reaction was reached. In this trial the maximum increase amounted to something over 20 per cent. This phenomenon is probably of little importance in the design of bridges, since the concentrated loads on bridge floors are moving loads, or at least remain in one position only a very short time. However, it may be of considerable importance in building floors designed to carry heavy machinery or similar loads.

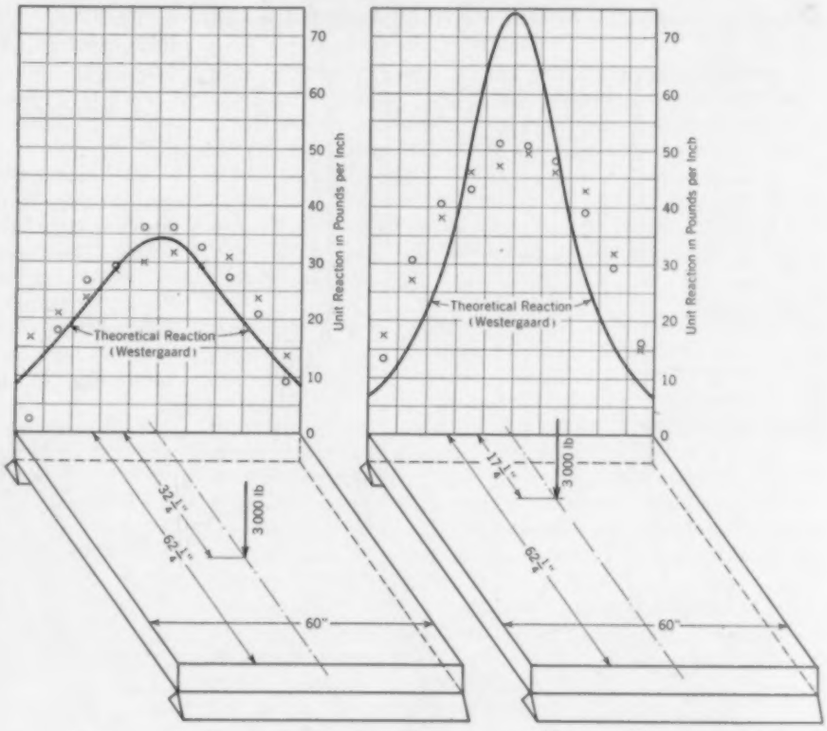


FIG. 8. AUTHORS' EXPERIMENTS COMPARED WITH THEORETICAL ANALYSIS
Close Agreement When Load Is at Midspan

Of the experiments conducted in this project the bulk were to determine the effective width for the reaction, and to discover if possible the laws controlling its variation (Fig. 6). Within the range of the dimensions covered by the 20 slabs studied to date, there appear to be two factors that influence the effective width for maximum shear stresses—the thickness of the slab and the distance of the load from the edge of the slab. Neither the span length nor the total width of the slab appear to exert an appreciable influence on the effective width (Fig. 7).

For any given thickness of slab, the effective width is a minimum for loads placed near the edge of the slab and increases approximately in a straight line ratio as the load is moved toward the longitudinal center-line, reaching a maximum value at some distance from the edge that is a function of the thickness and independent of the total width of slab. An empirical relationship which appears to fit the data satisfactorily has been devised by Professor Elbin as follows:

$$e = 0.75 + 0.1t + x \quad \ldots\ldots\ldots \quad [1]$$

in which e = effective width, in feet

t = slab thickness, in inches

x = distance from edge of slab to load, in feet, with a maximum value of $1 + 0.2t$

This empirical expression is offered tentatively until something more rational is evolved.

In any research of this general character, the maximum benefit from the experimental studies can be attained only when a rational theory can be found which is satisfactory in the light of the experiments. Such a theory permits an enlargement of the scope of the study and lends confidence in the application of the results to cases outside the range of the experiments. The only published analytical work directly applicable to this problem with which I am familiar is that by Dr. Westergaard, already mentioned. In this study he analyzed a thin slab of infinite transverse width. The analysis dealt with moment distribution under one, two, or four loads placed near the center of the span, either concentrated or distributed over a relatively small area. He also presented formulas for calculating the distribution of the reactions when a single load is placed at any point on the longitudinal center line of a slab, including points very close to the abutment under consideration.

Acting for the Engineering Experiment Station, Dr. Holl of the Mathematics Department of Iowa State College has utilized the same type of theory to analyze slabs of more specific dimensions, that is, those having finite width as well as length. His analysis gives the same maximum concentration of reaction at the center line as does that of Westergaard, but the unit reaction is somewhat higher for other points along the support. This is because the area under the distribution curve represents the total reaction on the finite width; whereas according to Westergaard this curve represents the distribution over an infinite width. Since it is the maximum concentration of reaction that governs the effective width, there is no practical difference in the two analyses. Also, Dr. Holl has analyzed the case of loads placed eccentrically with reference to the longitudinal center line. These mathematical analyses, together with more complete experimental data, will be published in the near future in the bulletin of the Iowa Engineering Experiment Station.

An attempt to correlate the distribution of reaction as measured in these experiments with that calculated by the analyses of Westergaard and Holl reveals a very wide divergence between the actual and the theoretical maximum unit reaction, when the concentrated load is placed near one support of the slab. This divergence diminishes rapidly as the load moves away from the support and disappears entirely when it reaches midspan (Figs. 8 and 9). Roughly it may be said that the theoretical analyses are adequate for calculating the reaction of a slab when the concentrated load is placed in the far two-thirds of the span but are not satisfactory when the load is in the near one-third of the span.

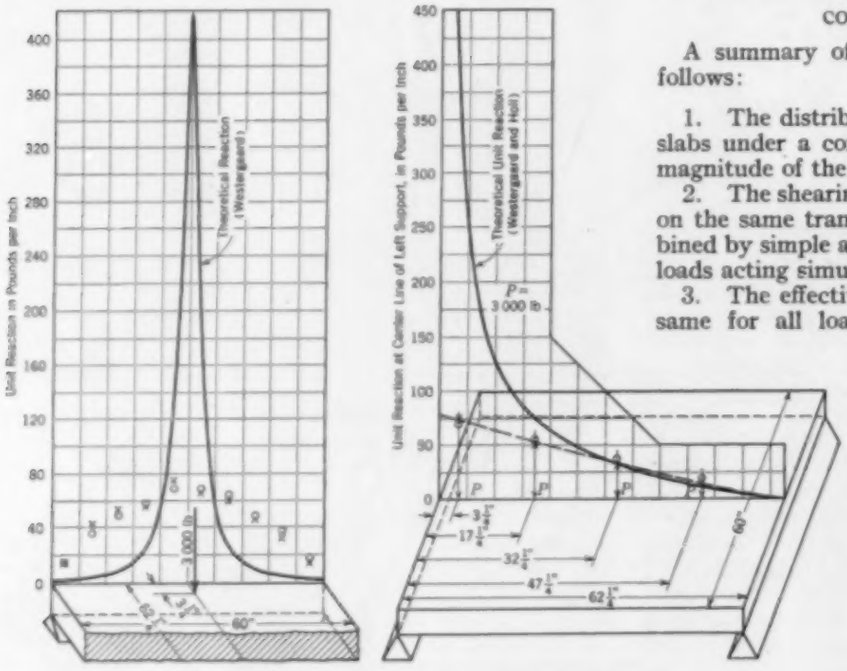


FIG. 9. COMPARISON BETWEEN TEST AND THEORY
Showing Wide Divergence When Load Is near Support

CONCLUSIONS SUMMARIZED

A summary of the results of this research to date follows:

1. The distribution of maximum shearing stresses in slabs under a concentrated load is independent of the magnitude of the load.
2. The shearing stresses produced by individual loads on the same transverse element of a slab may be combined by simple addition to obtain stresses due to all the loads acting simultaneously.
3. The effective width may safely be considered the same for all loads placed on the same longitudinal element of a slab.
4. The effective width for maximum shear stresses is dependent on the distance from the edge of the slab to the load and on the thickness of the slab. It may be expressed tentatively by the empirical formula, Equation 1.
5. The analytical determination of effective width for shearing stress at the support, by the thin-plate theory, does not agree satisfactorily with the experimental values when the load is located within the near one-third of the span.

Models Aid in Designing Irrigation Wasteway

Laboratory Tests Determine Construction Plans for Multiple-Weir Drop

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and A. W. NEWCOMER, ASSOC. M. AM. SOC. C.E.
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ANOTHER example of the use of models in the development of irrigation structures for a New Mexico irrigation project is here explained. A previous example—the tests made to determine the profile for the Cochiti Diversion Weir on the same project—was described in the January 1933 number. Faced with the necessity of dropping 400 cu ft per sec of water 86 ft to the Rio Grande, the project engineers felt that neither an open lined channel nor a closed conduit would safely handle the high velocities and absorb the great energy of the water. With no precedent to guide them, they designed a wasteway consisting of an open lined channel built on approximately a 12 per cent grade and crossed at frequent intervals by weirs. To determine the shape, height, and spacing of the weirs, a series of 20 models was tested in the hydraulic laboratory at the University of New Mexico. This article describes the tests, states the conclusions drawn from them, and explains such modifications of the design as were made during actual construction.

IN constructing the Belen Canal of the Middle Rio Grande Conservancy District, it was found necessary to provide a wasteway that could safely lower heavy discharges from a point high on the hillside to the valley floor and thence return the water to the river. At the same time, a supply of 100 cu ft per sec had to be delivered to the Sabinal Lateral at a point three-fourths of the way down the slope. It was desired to utilize a site where the 86 ft of vertical drop could be obtained in a rather short distance—about 700 ft—without curving the alignment or moving excessive dirt. The site selected is shown in Fig. 1.

Since the quantity of water might change rapidly from zero to 400 cu ft per sec, and since the maximum flow might continue over a long period, an open lined chute or closed conduit was considered entirely inadequate. It was believed that the resulting high velocities of 75 ft per sec would be extremely difficult to control; that any attempt to safely destroy kinetic energy equal to 4,000 hp in a single structure would be hazardous; and that only at great expense or possibly by using a second parallel conduit could 100 cu ft per sec be diverted directly from the wasteway at a point three-fourths of the way down the hill.

Preliminary designs following precedent produced no plan of a single structure that could be utilized with confidence. Inasmuch as rough estimates indicated that at least $30,000 would be needed even for these doubtful plans, it was decided to make experiments on a type of structure for which there was no precedent.

As shown in Fig. 2, the design which was evolved consisted of a concrete lined channel of uniform cross section having a length of 675 ft and a slope of 11.85 ft per 100 ft. At 42-ft intervals it contained V-shaped weirs 5 ft high with level crest and the point of the "V" upstream. The spacing of the weirs was chosen such that the crest of one weir would be at the same elevation as the base of the next weir above. This gave a drop increment of 5 ft and a minimum depth of water cushion equal to the head on the crest of the weir. The thickness of the concrete in the bottom of the channel was to be 6 in. and the thickness of the slopes, 4 in. The weir walls were to be 8 in. thick.

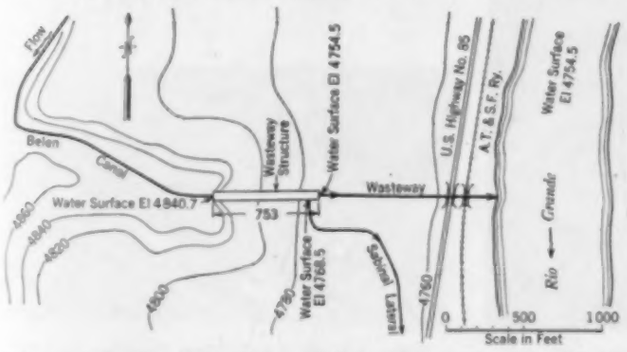


FIG. 1. SITUATION MAP SHOWING LOCATION OF SABINAL WASTEWAY STRUCTURE AT END OF BELEN CANAL, MIDDLE RIO GRANDE CONSERVANCY DISTRICT

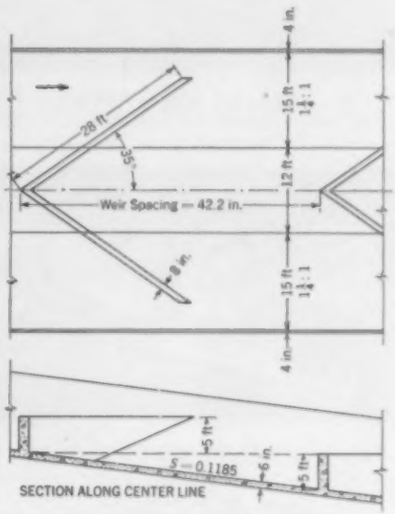


FIG. 2. DETAIL OF MODEL FOR TYPE AA WEIR, OF 5-FT HEIGHT
This Plan Was Slightly Modified in Construction

At the bottom, the wasteway was to terminate in a standard drop structure with a head of slightly over 6 ft, such as had been developed by the District for sandy soils and had proved entirely satisfactory. The standard drop structure consists mainly of a long V-shaped crest pointed upstream and spilling a thin sheet of water into a sump about 2 ft deeper than the normal canal depth.

The turnout structure for the Sabinal Lateral was designed to consist of a square slide gate with appropriate transitions, set behind the third weir from the bottom, which has a crest elevation almost 2 ft higher than the required water surface in the lateral.

Laboratory tests on hydraulic models of such a structure were undertaken with the assistance and facilities of the University of New Mexico at Albuquerque. A model ratio of 1:15 was selected. To similate 400 cu ft per sec in the prototype, 0.46 cu ft per sec was required in the model, which flow was carefully maintained during the tests except when it was increased to note the effect

Type AA Weirs, 5 Ft High

Type G Weirs, 11 Ft High, with Open Blunt Nose Downstream

Type E Weirs, 5 Ft High

Type AA Weirs, 7 Ft High

Type A Weirs, 5 Ft High, with Blunt Nose Upstream

Type E Weirs, 8 Ft High

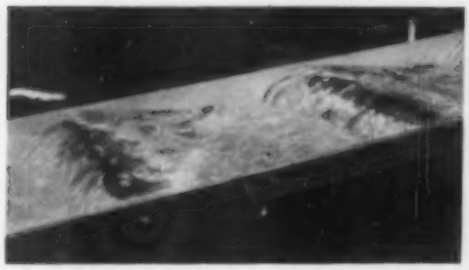
Type AA Weirs, 8 Ft High

Type F Weirs, 5 Ft High, Spacing 33.8 Ft

Straight Weirs, Type C, 8 Ft High

Model Ready for Test

Type G Weirs, 7 Ft High



Straight Weirs, Type C, 10 Ft High

Type J Weirs 2.8 Ft High, Spacing 23.8 Ft

WASTEWAY MODELS IN OPERATION

Discharge, 0.46 Cu Ft per Sec; and Spacing of Weirs, 42.2 Ft Except as Noted

of overloading the structure. The model was constructed with a base width of $9^5/_8$ in.; a depth of $9^5/_8$ in.; a horizontal width of side slopes of 12 in.; a top width of 2 ft $9^5/_8$ in.; a length of 20 ft; and side slopes of $1^1/_4$:1.

The model (Fig. 3) was built of timber and lined with white oilcloth, glued down with shellac, which proved very effective in preventing leaks even after numerous changes of the weirs, both in shape and spacing. The weirs were made of $^1/_2$-in. surfaced boards and were calked to prevent leakage. With a spacing of 2 ft 10 in., seven of them could be placed in the model.

STANDARD DROP STRUCTURE AT BOTTOM OF WASTEWAY COMPLETED
Workmen Placing Paving Above

Three general shapes were included: V-shaped with crest pointed upstream; V-shaped with crest pointed downstream; and straight, placed at right angles to the direction of flow. In addition, the V-shaped types were modified by cutting off the nose of the "V" to a width equivalent to 5 ft in the prototype. Tests were made on seven different types of weirs, and on each type at different heights. From these tests it was hoped to determine whether currents of high or increasing velocity would be developed or carried through the wasteway; whether a 5-ft increment of drop would be satisfactory; and what was the best length, form, and height for the weirs.

Sketches showing the form and dimensions of the seven types studied are shown in Fig. 3. In Table I will be found the shape and principal dimensions of the 20 weirs tested as well as the character of the resulting flow.

COMPLETED WASTEWAY STRUCTURE LOOKING DOWNSTREAM
For Scale, Note Man on Third Weir from Top

A group of laboratory photographs made to show the general flow characteristics that could be expected in the prototype are reproduced on page 596.

From the tests conducted on the 20 weirs listed in Table I, the following conclusions were drawn:

1. Increasing the depth of pool below the weirs was much more effective in securing a smooth and steady flow than any other factor.
2. In none of the weir models tested did the water show any tendency to accelerate as it passed down the structure.
3. Increasing the length of the weir crest helped to eliminate turbulence.
4. Sharp pointed weirs pointing upstream (Types AA and F) were only slightly superior to weirs with a blunt nose (Type A).
5. All V-types pointing upstream had better hydraulic properties than those pointing downstream or than

TABLE I. PROTOTYPE DIMENSIONS AND CHARACTER OF FLOW FOR THE 20 WASTEWAY MODELS TESTED
Discharge, 0.46 Cu Ft per Sec in Model, Equivalent to 400 Cu Ft per Sec in Prototype

Type	Height Ft	Drop Increment Ft	Spacing Ft	Angle with Downstream Center Line, Deg.	Crest Length Ft	Head on Crest Ft	Depth of Pool Under Weir Ft	Shape of Weir	Character of Flow
AA*	5	5	42.2	35	56	1	1.7	Point of V upstream	Fairly rough water
AA*	7	5	42.2	35	71	1.4	3.4	Point of V upstream	Smoother surface
AA*	8	5	42.2	35	78	1.3	4.3	Point of V upstream	Very smooth and satisfactory
A*	5	5	42.2	33	51	1.8	1.8	5-ft blunt nose upstream	More turbulent than 5-ft AA type
A	6	5	42.2	33	58	1.6	2.6	5-ft blunt nose upstream	Smoother than 5-ft A type
A	8	5	42.2	33	70	1.4	4.4	5-ft blunt nose upstream	Very smooth and satisfactory
C	5	5	42.2	90	24	2.9	2.9	Straight and normal to flow	Very rough and irregular
C*	8	5	42.2	90	32	2.4	5.4	Straight and normal to flow	Induced surges carried through
C*	10	5	42.2	90	37	2.2	7.2	Straight and normal to flow	Tendency to rotate in pools, but fairly smooth
E*	5	5	42.2	150	39	2.1	2.1	Point of V downstream	Hydraulic jump between weirs
E*	8	5	42.2	150	51	1.8	4.8	Point of V downstream	Surges increasing in magnitude
F	5	5	42.2	30	67	1.5	1.5	Point of V upstream	Smoothest of the 5-ft weirs
F	6	5	42.2	30	73	1.4	2.4	Point of V upstream	Very satisfactory
F*	5	4	33.8	30	67	1.5	2.5	Point of V upstream	Smooth and satisfactory
G	5	5	42.2	150	37	2.2	2.2	5-ft blunt nose downstream	Very turbulent; hydraulic jump between weirs
G*	7	5	42.2	150	46	1.9	3.9	5-ft blunt nose downstream	Jump removed, turbulent flow
G	8	5	42.2	150	51	1.8	4.8	5-ft blunt nose downstream	Slightly better than 8-ft E type
G	11	5	42.2	150	59	1.6	7.6	5-ft blunt nose downstream	Surges or vibrations still present
G*	11	5	42.2	150	59	1.1 and 5.1	7.1	Same except 5-ft nose open to depth of 4 ft	Better than with closed nose
J*	2.8	2.8	23.8	20	90	1.2	1.2	Point of V upstream	No advantage over higher weirs spaced farther apart

* Illustrated on page 596.

those extending straight across the channel, weir heights and drop increments being equal.

6. Moving the weirs closer together—keeping the slope of the channel and the height of the weirs constant—had the effect of increasing the depth of the pool

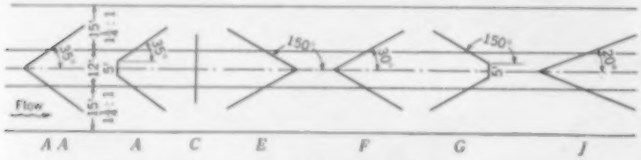


FIG. 3. THE SEVEN TYPES OF WEIRS TESTED
Type AA Selected for Construction

under the weirs and decreasing the drop increment, and obviously improved the structure.

7. Weirs pointing downstream showed a tendency to carry induced surges through the entire length of the chute, but they gave a fairly good water surface if raised to a sufficient height.

8. Weirs terminating in a sharp point and pointing downstream (Type E) gave a more turbulent flow than the blunt nosed ones, such as Type G.

9. Weirs built straight across the channel gave a fairly smooth water surface when raised to a sufficient height but had a tendency to pass induced surges down the chute.

10. A structure of this sort, using weir Type AA or A, will completely dissipate energy for any height of total drop.

11. Objectionable deposits of silt will not form in a structure of Type AA. Pea gravel placed in the model was carried through. The action of the water rolled it up the slope along the face of the weir.

12. For a weir similar to type AA, only a substantial change in length of crest, height of weir, drop increment, spacing of weirs, or depth of pool will cause any visible change in the way the structure functions. Increasing the design discharge of the model by as much as 20 per cent made but little difference in its action. It performed equally well or better when the flow was decreased below the design discharge.

As a result of the model tests, a design was prepared with weirs similar to Type AA. The channel was given a base width of 12 ft, a depth of 15 ft, and side slopes of $1\frac{1}{4}$:1. Its length was made 703 ft and its slope, 11.9 per cent. The weirs were given a height of 6 ft 6 in., a crest length of 68 ft, and a spacing of 40 ft 4 in. This gave a head of 1.5 ft on the crest of the weirs, a minimum pool depth of 3.25 ft under the weirs, and a minimum freeboard of 3.5 ft.

When the ground was opened it was found that the natural slope of the material was about 2.1, and the side slopes of the paving were changed to agree. This caused a slight change in the angle of the weirs, which was made 45 deg (Fig. 4), to maintain about the same length of crest. The weirs were designed as cantilevers extending from the floor of the channel, the paving of which was thickened to 8 in. directly under the weir walls. To accommodate the 40-ft commercial lengths of reinforcing steel, contraction joints and weirs were spaced 40 ft 4 in. apart.

PAVING SIDE SLOPES OF WASTEWAY
Dowels for One Weir in Place

The laboratory experiments indicated that additional freeboard could be obtained by lowering the crest of the weirs for a short distance back from the point of contact with the channel lining. In the actual structure this was accomplished by terminating the weirs where they had a height of 12 in. above the side-slope paving. Each weir pool is provided with two drain holes 6 in. square, one at the lowest point on each side of the weir wall.

Construction was completed in September 1934. The completed structure involved 20,000 cu yd of excavation, and 1,100 cu yd of concrete, and was completed at a cost of approximately $37,000. The model tests and the construction of the wasteway were accomplished under the direction of C. H. Howell, M. Am. Soc. C.E., chief engineer of the Middle Rio Grande Conservancy District.

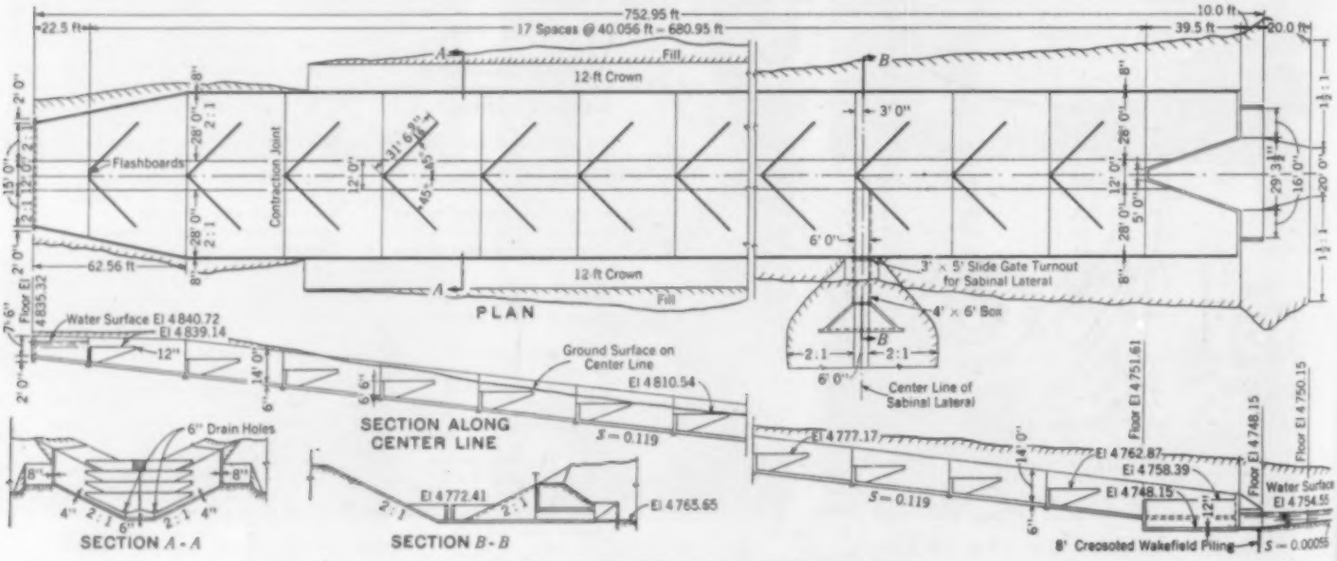


FIG. 4. WASTEWAY STRUCTURE AS CONSTRUCTED
Completed in September 1934

ENGINEERS' NOTEBOOK

From everyday experience engineers gather a store of knowledge on which they depend for growth as individuals and as a profession. This department, designed to contain practical or ingenious suggestions from engineers both young and old, should prove helpful in the solution of many troublesome problems.

Sand Traps in a Power Canal

By T. K. MATHEWSON, M. AM. SOC. C.E.
SANTA ANA, CALIF.

IN South American hydro-electric developments, special desilting devices are necessary because of the great quantities of rock, gravel, sand, and silt brought down by the melting snow. There mid-summer comes in January during a long rainless period in which the ice and snow on the high cordilleras melt and appear in torrential streams. Mid-winter comes in July, when the precipitation falls in the form of snow and is stored on the slopes of the Andes. The run-off during this period is from springs, clear and constant.

An existing power plant has a row of hoppers near the forebay to trap the sand. In the bottom of each hopper (Fig. 1) is a hardwood clean-out valve which controls the flow; but because of the large heads and high velocities the valves must be throttled down to such a small opening that they are easily plugged by small bits of trash. Then the hoppers fill with sand, necessitating a troublesome job of shoveling out the hoppers and cleaning the valves.

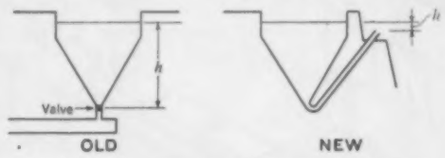


FIG. 1. TWO TYPES OF SAND TRAPS ON A SOUTH AMERICAN POWER DEVELOPMENT

At a new power plant the stones are kept out of the canal at the intake in the river by the usual arrangement of a deep sluice gate in the weir and a skimming wall. The upper end of the canal was given enough grade to carry gravel to a row of hoppers above the plant. In the new hoppers it seemed best to eliminate valves entirely, with all their troubles. The desired rate of discharge was obtained by reducing the head, and 4-in. sewer pipes were utilized to bring the discharge up to the desired elevation (Fig. 1). There is no throttling anywhere, and no excessive velocity.

These hoppers were only intended to eliminate gravel and coarse sand, as the stones were taken care of at the intake in the river. Nevertheless, I have tossed jagged stones as big as my fist into them, and in every case the stones were spit out at the end of the pipe in four or five seconds, indicative of its complete operation.

At the time of low water the river is clear and the whole flow is by-passed around the hoppers to save water. When the river is in flood and full of sediment there is also plenty of water to waste in sluicing operations.

Reinforced Hollow Tile Proposed for Building Walls

By R. E. RUBINS
ASSOC. M. AM. SOC. C.E.
ASSISTANT RESIDENT ENGINEER INSPECTOR, PUBLIC WORKS ADMINISTRATION, GLENCOE, MINN.

ALTHOUGH for many purposes hollow tile has distinct advantages over other materials, it has not been used as universally as it could be because, strangely enough, it is not heavy enough on the one hand and it is too heavy on the other. Its weight, and consequently its cost delivered, in competition with lumber is distinctly to its disadvantage for the building of small structures. On the other hand many building departments will not permit its use for backing up a stone veneer face in a steel or concrete skeleton building because, owing to insufficient weight, the center of gravity of the wall is not thrown far enough back on the supporting shelf to provide unquestioned stability. However, it is believed that the method here outlined will eliminate these disadvantages and open up a considerably wider field for the use of tile.

THE METHOD

The idea is so to reinforce hollow-tile construction with pencil rods that a 4-in. wall will act as a vertical slab between story heights. This really means that all buildings would be designed as articulated frame structures but that in small buildings each floor slab would be considered to act as a horizontal diaphragm reinforcing the building as a box, as in ordinary frame construction. This might affect the design of openings in the floor near the walls and, in the case of a wood joist floor, would require additional bracing on the sides where the wall is parallel to the joists. In Fig. 1 is shown the application of the method to a finished exterior wall, and in Fig. 2, a suggested means of handling such openings as doors and windows. It should be noted that a sill or lintel can be formed by a built-in-place brick beam, and that, if the floor construction requires it, a reinforced brick beam can be built opposite the floor levels. Whatever the form of tile used, it must have a recess vertically up the center of

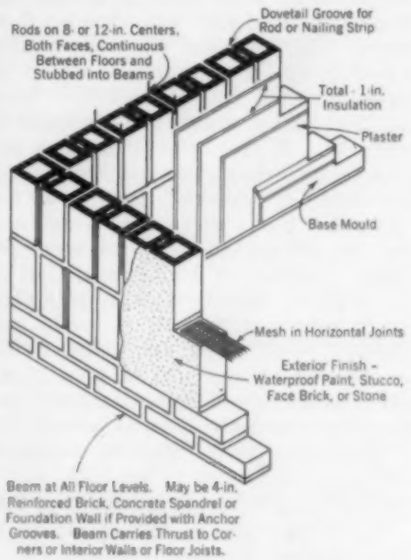

Rods on 8- or 12-in. Centers, Both Faces, Continuous Between Floors and Stubbed into Beams
Dovetail Groove for Rod or Nailing Strip
Total - 1-in. Insulation
Plaster
Base Mould
Mesh in Horizontal Joints
Exterior Finish - Waterproof Paint, Stucco, Face Brick, or Stone
Beam at All Floor Levels. May be 4-in. Reinforced Brick, Concrete Spandrel or Foundation Wall if Provided with Anchor Grooves. Beam Carries Thrust to Corners or Interior Walls or Floor Joists.

FIG. 1. TYPICAL DETAIL OF WALL CONSTRUCTION

each face to receive the rods through the staggered courses, and this groove should, if possible, be of dovetail form. The slots can also be used conveniently for securing nailing strips where required. The size of the unit illustrated is roughly 8 by 8 by 4 in. because such dimensions are convenient for matching with brickwork.

Several differences between this method and ordinary tile construction practice should be noted. It is essential that all work be laid up in portland cement mortar, possibly tempered with 10 per cent lime; this will give a considerably stronger wall although it will add slightly to the cost. Laying the tile on end presents no particular difficulties provided it is made with some sort of double web diaphragm so that each tile will get full bearing on the unit below. It is believed that some sort of a fine mesh must be used in the horizontal joints to ensure even spreading of the mortar. Tests should be made to determine what efficiency such a joint has in compression.

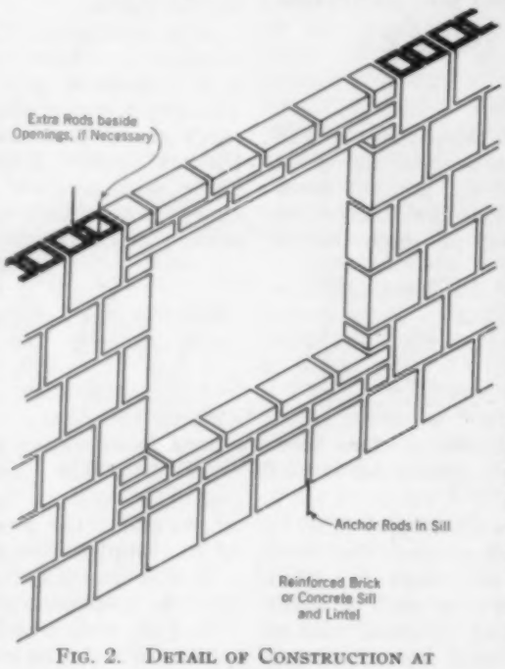


FIG. 2. DETAIL OF CONSTRUCTION AT AN OPENING

SMALL BUILDINGS

Lumber and cement blocks have practically crowded tile out of use for small buildings and residences. Cement block has the advantage that it is usually made in the locality where it is used, so that transportation costs are not an appreciable item. However, the block is often very poorly made; it is subject to excessive shrinkage and cracking; and it is not easily adapted to any but crude construction. It is almost always preferred to tile or brick for reasons of economy alone.

For the average inexpensive dwelling, there is no question but that lumber is the logical material under present conditions, even though it is not fireproof and requires continual maintenance. However, it seems inconsistent with good judgment to build a $20,000 residence faced with beautiful brick and backed up with wood. If the usual tile construction is employed for backing up, it increases the wall thickness to 13 in. and raises the cost alarmingly. It is believed that the method here proposed will overcome the difficulty almost entirely. If the basic structural material of the wall can be held down to a thickness of 4 in., that is, about one-half or one-third that now required, any slight difference that may remain in cost will be far outweighed by the advantages of a fireproof, permanent construction. Further, it would seem that the tile itself might be so placed that it would form a pleasing exterior. If a brick veneer were used, it would need to be only 2 in. thick. Such basic tile construction will lend itself excellently to any sort of plaster or stucco covering if there is some undertreatment to render the wall waterproof, particularly in areas subject to heavy frost.

This same general method is applicable to the building of one- and two-story garages, warehouses, and stores, particularly where ordinances require fireproof methods. The competition in such cases is with cement block, unreinforced tile, and common brick, all of which form walls from 8 to 13 in. thick. The saving is obvious.

The time-honored method of building the exterior walls of a stone or brick-faced steel or concrete frame building is to lay up masonry to a thickness of 16 or 18 in. and of such weight that the center of gravity falls well back on the supporting shelf. The outside edge of this shelf is usually 4 or 5 in. back from the face of the wall so that hollow tile of ordinary thickness is too light for stability. It seems ridiculous to put up a wall sufficiently heavy to support several stories by itself, and then to add additional section to the columns to carry the excess weight. And there is some doubt in many minds as to whether such construction offers sufficient resistance against the suction action of the wind, which tends to make the wall fall outward. The proposed design would secure stability with a 2-in. stone veneer and 4-in. bond stones backed up with 4- or 6-in. tile reinforced between floor spandrels. Of course it would be necessary to groove the back of the bond stone for the rods and to anchor the facing securely in place so that the stress would be transmitted to the rods, but this inconvenience would be much more than outweighed by the saving in material and weight. It might be necessary under certain exposures to add insulation and waterproofing. The use of masonry for insulation only is hardly the most economical of practices.

Further, it seems possible that a structural slab could be relied upon to resist wind action better than a wall which might shrink entirely away from the spandrel above it. Certainly the tile construction would resist the formation of cracks due to shrinkage or settlement better and should withstand earthquake disturbances more efficiently.

DEFINITE BENEFITS

Its advantages may be summarized as follows:

1. It permits the building of fireproof walls for dwellings and small buildings at a cost at present considerably below that of tile or brick, comparable with that of cement block, and not much more than that of lumber.
2. It can cut the weight of exterior curtain walls in stone- or brick-faced buildings practically in half and at the same time give considerably more rigidity.
3. In any type of structure, it should have less tendency to show shrinkage or settlement cracks than plain brick, block, or tile masonry.

On the other hand, it must be recognized that additional care must be exercised in laying the units to ensure proper bond for the rods and to protect them from rust. Also it is more difficult to lay tile with the cells vertical than horizontal.

It has always seemed to me that there is much room for improvement in ordinary masonry construction. Materials have improved; methods of handling them have improved. But still the units are laid up, block by block, as they have been for centuries. The only change seems to be a gradual deterioration in the nicety with which the work is done. It seems that this proposed system has a wide applicability to practically all types of masonry walls and that it may bring back

the conception that a small structure can survive more than 40 years of useful service. It is believed that the use of reinforcement in connection with hollow tile is new. Since, as far as I know, the method has never been tried in practice, it will require some experimental work before it can develop into a useful construction method susceptible of accurate analysis. Probably the design method will be very similar to that for reinforced concrete.

Nomograph for Flow of Water in Cast-Iron Pipes

By JAMES R. GRIFFITH, M. AM. SOC. C.E.
PROFESSOR OF STRUCTURAL ENGINEERING
OREGON STATE COLLEGE

IN 1925 Wegmann and Aeryns presented a new formula for the flow of water in clean cast-iron pipes. This formula,

$$V = 182.5\ R^{0.723}\ S^{0.539}$$

was later adopted by the Cast-Iron Pipe Research Association and published, with computed tables, in their *Handbook of Cast-Iron Pipe*. Although Wegmann and Aeryns published a chart for the solution of their equation, I have long been a believer in nomographic charts for a solution as compared to any other type of chart or computed tables. This is especially true in the making of first estimates where all the accompanying changes for different assumptions must be known.

Hydraulic flow formulas of the constant-exponential type are ideally suited to nomographic solution. With any two known or assumed conditions given, a multitude of information is available at a glance. There have been many nomographic solutions published of other flow formulas, especially for the well-known fundamental Chezy formula. It is believed that the nomograph presented in Fig. 2 is the first that has been published for the Wegmann-Aeryns formula.

A straight line intersecting all scales will satisfy all conditions of the formula. A solution is shown in Fig. 1, assuming a loss of head of 0.49 ft per 1,000 ft of pipe, and a pipe diameter of 42 in. When these points are connected, it is found that the pressure loss is 0.21 lb per sq in. per 1,000 ft of pipe; the velocity, 2.7 ft per sec; the velocity head, 0.12 ft; and the discharge, 11,500 gal per min, that is 26 cu ft per sec.

To illustrate the comparative accuracy of the chart and to check this computation, the following information, for the assumed conditions, has been taken from the

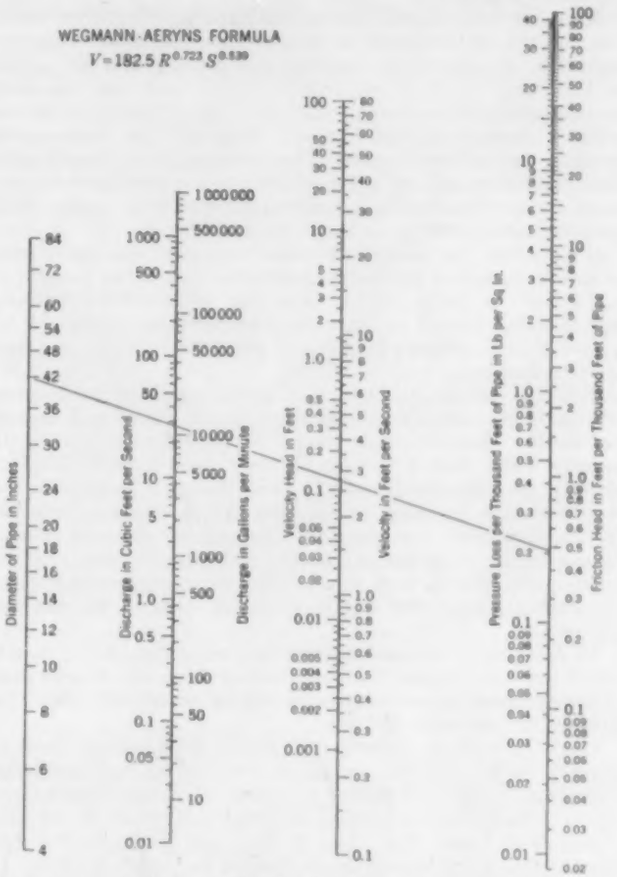

WEGMANN-AERYNS FORMULA
$V = 182.5\ R^{0.723}\ S^{0.539}$
Diameter of Pipe in Inches
Discharge in Cubic Feet per Second
Discharge in Gallons per Minute
Velocity Head in Feet
Velocity in Feet per Second
Pressure Loss per Thousand Feet of Pipe in Lb per Sq In.
Friction Head in Feet per Thousand Feet of Pipe

FIG. 1. FLOW OF WATER IN CLEAN CAST-IRON PIPE

Handbook of Cast-Iron Pipe: velocity equals 2.73 ft per sec, and discharge equals 11,110 gal per min.

By slide rule,
Pressure loss = 0.212 lb per sq in. per 1,000 ft of pipe
Velocity head = 0.116 ft
Discharge = 24.8 cu ft per sec

A table of computed values may give a higher degree of precision than a chart. However, the table is open to the objection of possible errors in individual computations and the difficulty of interpolation.

Our Readers Say—

In Comment on Papers, Society Affairs, and Related Professional Interests

Inter-Continental Highway Economically Unsound

TO THE EDITOR: The article, "From Panama to the Rio Grande by Highway," by Mr. James in the symposium on "An Inter-Continental Highway System," in the September issue, is of particular interest to me. However, I cannot agree with Mr. James that such a highway would be of economic value.

Although an increase in tourist travel might result from the construction of such a highway, it hardly seems possible that that alone would justify economically its construction and proper maintenance (an extremely important factor). Furthermore, there would be little use for such a highway. At present there is practically no exchange of commodities between the various countries of Central America or even between those of Latin America. The general economic system of all the countries south of Mexico City is based on practically the same conditions. In the interior are produced certain raw materials, usually coffee or minerals, for which a market can be found only in Europe and the United

States. These countries import manufactured goods, along with a certain amount of wheat products. Even if there were successful diversification of crops, the problem of distribution would not be materially changed.

What the countries in Central and Latin America need, therefore, is the development of lines of communication, railways, highways, or waterways, between the interior and the coasts so that their products may reach a port and that manufactured products from Europe and the United States can be distributed throughout the interior. None of the countries in question has sufficient capital or borrowing capacity, even in good times, to finance all the needs of these really vital lines of communication. Therefore, it would be a grave error to burden these countries with a debt for an international highway.

It is obvious that none of the import or export business referred to can be benefited by this international highway or handled on it. Laredo is fairly distant from the main coffee consuming and blending centers, or from the manufacturing centers of the United States, without considering the 3,000-mile journey from there to Panama.

It is true that in some countries, particularly Guatemala, progress has been made in the construction of well graded and drained dirt roads. However, these are for the local development of the country which such a through highway could not affect, except through possible tourist travel. Such travel is practically impossible during the rainy season, from May to October, inclusive. In fact November, December, and January are the only delightful months in which tourist travel is really comfortable, and it is questionable whether such travel during this period would warrant the capital expenditure and maintenance costs of the proposed highway.

To date history does not support the theory that the freedom of travel and interchange of ideas, resulting from the development of international communication or tourist travel, will make for friendly international relations.

In the matter of maintenance careful consideration must be given to the fact that the geologic structure of most of this terrain is of volcanic origin and subject at present to volcanic disturbances. The rock formations are broken, distorted, and extremely unstable. Heavy floods occur from time to time, and the flashy mountain streams then carry large volumes of water and much detritus.

Government work quite generally suffers from lack of appropriations for maintenance, and this is especially true of Latin America where, with meager revenues and perennial difficulty in meeting necessary budget expenditures, there is a tendency to eliminate items for maintenance. Even should the United States arrange to furnish all the money necessary for the initial cost by loaning it to the countries in which the construction is to be carried out, this is of doubtful wisdom as it would involve adding to their already onerous debt burdens.

FRED LAVIS, M. Am. Soc. C.E.
Consulting Engineer

New York, N.Y.
October 10, 1934

Design of Bonneville Power House

TO THE EDITOR: A few comments on Mr. Torpen's article, in the September issue, may be of interest. The Bonneville power development is of the low-head type, uncommon in the West. It differs from a high or medium-head development in both machinery and structures. The power house substructure consists largely of water passages separated by dividing walls of minimum width. Its dimensions are controlled by the space required for these water passages, which must be large because the velocities are limited. Therefore the economy of design is dependent on permissible velocities. The substructure in a higher head development is principally a supporting foundation for machinery and for related manufactured parts.

Low-head turbines require adaptability to a wide range of heads. They necessarily operate at relatively low speeds, entailing correspondingly high cost of generators. High-velocity draft tubes require low elevation of the turbine runners with reference to minimum tailwater in order to avoid cavitation. The use of high velocities in the water passages, to secure economy in the structure, is limited because of hydraulic friction and disturbance in flow with its resultant turbine vibration. In large turbines and water passages, friction becomes almost negligible and disturbance and vibration can be eliminated to the extent that it is possible to guide the water in its path of flow. Correct guiding of the water is easy in the ideal scroll case of a high-head turbine where all water enters tangentially on one side of the turbine. But it is very difficult in a low-head installation. In fact it is not done in correct practice, and recourse is had to lower velocities instead.

In order to justify higher velocities and consequent economy in the length of the Bonneville power house, which is greatly restricted by natural features, an effort is being made to develop, by research, a design for a scroll case in which the water will be properly guided. Studies are also being made in an effort to improve the draft tubes.

The selected Kaplan, or governor adjustable-blade, propeller type of turbine, as compared with the fixed-blade type, is much better adapted for high efficiencies over a wide range of heads and loads; it is subject to less vibration at partial loads and abnormal heads; it can be operated at considerably above the normal power rating for short periods of time without important loss in efficiency; it has considerably greater output when operating under reduced heads; and it is more stable when operating at partial loads. Under the wide range of heads at Bonneville, it utilizes a smaller generator than is required for the economical operation of a fixed-blade turbine at times of maximum head and minimum water. This reduction in size of generator tends to offset the greater cost of this type of turbine.

These are some of the fundamental considerations which entered into the solution of the major problems met with in the design of the Bonneville power house.

L. F. HARZA, M. Am. Soc. C.E.
Consulting Engineer

Chicago, Ill.
September 26, 1934

Temporary Hinges for Fourth Street Viaduct

DEAR SIR: The question of priority of construction of three-hinged arch bridges has overshadowed discussion of the observations, data, and questions raised by the behavior of the arch hinges and abutments in the Fourth Street Viaduct. I had hoped that others with experience in the use of temporary hinges would contribute discussion of general professional benefit. It is a matter of regret to me that in my article, in the April issue, in which I stated that this type of construction was first used for the Fourth Street Viaduct, I did not use the phrase, "so far as is known to us." In January 1927, a study of available indexes and textbooks failed to reveal the fact that bridges of the temporary-hinged type had been built in the United States. Furthermore, no specific information of the sort given in my article was discovered.

In the September number, C. M. Spofford, M. Am. Soc. C.E., calls attention to the Hampden County Memorial Bridge across the Connecticut River at Springfield, Mass., constructed in 1920–1922. The interesting letter by Mr. Reed, in the June issue, directs attention to *Elastic Arch Bridges* by C. B. McCullough and E. S. Thayer. This excellent text, which was published in 1931, states that the Grants Pass Bridge over the Rogue River in Oregon was swung on November 25, 1930 (page 331).

Design for the Fourth Street Viaduct was begun in January 1927 and the arch ribs were decentered December 22 and 23, 1930. Thus both the design and construction antedated the publication of *Elastic Arch Bridges*. As to date of decentering, however, the Fourth Street Viaduct appears to be the third. It is possible, however, that there are other bridges of that type that have not yet been mentioned.

LEON BLOG, Assoc. M. Am. Soc. C.E.
Assistant Chief Structural Engineer, Division of Bridges and Structures, City of Los Angeles

Los Angeles, Calif.
September 21, 1934

SOCIETY AFFAIRS

Official and Semi-Official

Plans for Annual Meeting Developing

PREPARATIONS for the coming Annual Meeting to be held in New York, January 16–19, 1935, are rapidly being perfected. Each successive meeting, while based on a skeleton plan that is more or less standard, nevertheless differs widely in details. Thus the coming Annual Meeting will vary from previous meetings in a number of aspects.

Attendance is of course always one of the unknown factors, but if the demand for the presentation of valuable technical material can be taken as a guide to the probable attendance, this Annual Meeting promises to outstrip those of the past few years. In fact, all but two of the ten Technical Divisions have definite programs in preparation.

This situation, while gratifying, presents certain complications. The number of sessions to be scheduled so taxes the meeting-room facilities available at Headquarters that the committee in charge has been forced to set aside the afternoon of the opening day, Wednesday, January 16, for simultaneous sessions of Technical Divisions. This is a complete revision of the usual plan of holding a general technical meeting of the whole Society at that time. In view of the popularity of the technical sessions held by the Divisions and the great variety and appeal of the programs to be presented, this innovation will be viewed as an improvement by many of those who regularly attend these meetings. The following Technical Divisions have scheduled programs, either for Wednesday afternoon or for Thursday morning and afternoon: City Planning, Construction, Highway, Power, Sanitary Engineering, Structural, Surveying and Mapping, and Waterways.

Social events have always formed an important part of the Annual Meeting. The committee is actively at work on these features, which add so much to the popularity of the occasion. One entirely new plan has already been definitely adopted, so that announcement can be made of it at this time—far enough in advance for members to make the necessary preliminary arrangements. This consists of a short trip to Bermuda following the Meeting, open to members and guests. For this excursion, which is on the all-expense basis, special rates are offered. To avail themselves of these, however, members are requested to notify the Secretary of their intention to go at as early a date as possible. The party is scheduled to embark at 3 o'clock on the Saturday afternoon following the Annual Meeting and to return to New York City on the morning of the following Friday. After spending all day Sunday at sea, the party will arrive at Hamilton, Bermuda, early on Monday. This will give almost three days for delightful sight-seeing and other diversions before leaving Wednesday afternoon for the return trip. Accommodations will be on the *Monarch of Bermuda*, one of the finest liners in this or any other passenger service in the world.

Announcement of further details of the Annual Meeting and the special excursion will be made as they are developed. Additional particulars will be found in the December number of CIVIL ENGINEERING and complete information will be given in the official program, to be printed in the January 1935 number.

Society Meeting, October 17, 1934

AS DEMANDED by the Constitution, an official meeting of the Society was held in New York, N.Y., on the evening of October 17, 1934. It was called just prior to the regular October meeting of the Metropolitan Section, with Director Trout in the chair. Announcement was made that the particular purpose of having Society meetings in March and October is to provide for the counting of votes on amendments to the Constitution. There being no amendments in process and no other business, the meeting was adjourned.

Board of Direction Meeting—Secretary's Abstract

ON October 1 and 2, 1934, the Board of Direction met at the Edgewater Beach Hotel, Chicago, Ill., with Harrison P. Eddy, President, in the chair; and present George T. Seabury, Secretary; and Messrs. Ammann, Barbour, Black, Crocker, Dewell, Enger, Etcheverry, Gregory, Hammond, Hoffmann, Hogan, Horner, Jonah, Lupfer, McDonald, Noyes, Perry, Reed, Riggs, Sanborn, Sherman, Stevens, Trout, and Wilkerson.

Approval of Minutes

The Board approved the minutes of its meetings held on July 9 and 10, 1934. Also, the Board approved the minutes of the meeting of the Executive Committee held on October 1, 1934, having to do with matters of a financial or administrative character, and adopted these actions as the actions of the Board.

Bureau of Standards

A resolution was adopted referring to the retrenchment policy of the Federal Government which effected a drastic curtailment in the activities of the Bureau of Standards. The resolution urged upon the appropriate committees and members of the Congress, the making of an appropriation for the coming fiscal year to provide adequately for fundamental scientific and engineering research and development in the science of measurement at the Bureau of Standards.

Politics and Engineering Appointments

Continuing the position taken at the time of the summer meeting of the Board when a resolution was adopted and forwarded to the President of the United States and the Secretary of the Interior protesting against the entrance of political considerations into the selection of engineers for any position, the officers of the Society were instructed to continue observations as to the extent to which this practice is being continued or has been rectified by the revoking memorandum issued by the Secretary of the Interior.

Governmental Competition with Private Practice

Having learned of instances in which a Federal bureau has entered into contract with a municipality for the rendering of engineering service, the Board appointed a committee of five Board members, of which Past-President Hammond is chairman to determine to what extent the Bureau in so doing is exceeding its congressional authorization, and to take the matter up with the appropriate Federal authorities.

Committee on Legislation

Since experience has shown that the Committee on Legislation established by the By-Laws of the Society is impracticable the By-Laws were amended to discontinue that committee. Like all committees established under the Administrative Department of the Society, the personnel of this committee had been distributed among various localities relatively close together, and yet not sufficiently close for efficient cooperation. Furthermore, in recent years, the Board as a whole has operated in large part as would a Committee on Legislation. The function of such a committee is thus transferred back to the Board itself.

Appointees to the Executive Committees of Divisions

The following appointments were made on the executive committees of the Technical Divisions, for 5-year terms:

City Planning Division	Hugh E. Young
Construction Division	A. E. Horst
Engineering-Economics and Finance Division	F. H. Fowler
Highway Division	Leslie G. Holleran
Power Division	W. P. Creager
	S. H. Woodward (to fill vacancy)

EIGHTY-SECOND ANNUAL MEETING, January 16–19, 1935, in New York, N.Y., at Society Headquarters

Structural Division . . . . . W. M. Wilson
Surveying and Mapping Division . . . . . Charles J. Tilden
Waterways Division . . . . . William G. Atwood

Appointments to the executive committees of the Irrigation Division and the Sanitary Engineering Division were referred to the Executive Committee with power.

South Carolina Section

The Board approved the formation of a South Carolina Section of the Society by members resident in that state. Temporary officers of that Section were elected at an organization meeting held at Columbia, S.C., during the summer, as follows: Robert L. Sumwalt, M. Am. Soc. C.E., temporary president; and Albert E. Johnson, Assoc. M. Am. Soc. C.E., temporary secretary-treasurer.

New Officers Invited to Board Meeting

In accordance with the procedure in former years, it was decided that those members who are to become officers of the Society at the time of the Annual Meeting in January 1935, will be invited as observers to attend the sessions of the outgoing Board of Direction.

Reinstatements

Realizing that because of economic or other conditions during the past few years, many members have relinquished their membership in the Society, a liberal policy with respect to reinstatements was adopted. Former members are to be informed of the new provisions as they may be applicable to their cases. Two methods of reinstatement are provided. In one the applicant will pay all charges outstanding and the dues for the intervening years. In consequence he will be entitled to receive all back publications available, to retain his original date of membership and his original date of exemption from dues. The other method permits reinstatement by the payment of all outstanding charges and of current dues in full, but with the remission of all other dues. In these cases no back publications will be forwarded, but the original date of membership will be retained. However, there will be an extension of the date of exemption from dues for a period equal to the number of years for which dues are not paid. Members whose connection with the Society ceased prior to 1930 may reenter only upon approval of a new application but without payment of a second initiation fee. Juniors incapable of reinstatement on account of age may reenter, if elected to Corporate Membership upon application, by the payment of the difference in entrance fees and of "prorated dues" for the year of reentrance.

Committee Reports

Reports were received from representatives of the Society on the United Engineering Trustees and the Engineering Foundation; and from the Executive Committee and Committees on Publications, Professional Conduct, Districts and Zones, Student Chapters, Juniors, and Public Education.

The Executive Committee reported on several financial and administrative matters; the Committee on Professional Conduct on three matters that had been referred to it, one of which was the advisability of adding a paragraph to the Code of Ethics. By action of the Board the proposed wording of such an additional paragraph was approved and ordered sent to the Corporate Members of the Society for confirmation. The Committee on Publications recommended that CIVIL ENGINEERING be sent regularly to one of the officers of each of the Student Chapters, and the Board adopted the recommendation. The Committee on Districts and Zones recommended that there be no change in the boundaries of Districts and Zones for the year 1935. The Committee on Student Chapters advised the Board that due to reorganization of the educational institutions within the State of Georgia, the University of Georgia had dropped its course in civil engineering and the Student Chapter there had been discontinued. On recommendation of the Committee on Accredited Schools the Board recognized the Utah State College as an accredited school in civil engineering.

Honorary Membership

The Board elected to Honorary Membership, Robert Ridgway, Past-President of the Society, and Milo S. Ketchum, former Vice-President of the Society.

New Engineering Societies

The Board received a report from its Special Committee appointed to study the organizations of engineers newly formed or in prospect of formation. The committee analyzed the objectives of the several organizations known to be in process of formation, comparing their objectives with the present human and public relations activities of the Society. The Board adopted the committee's recommendation that it be continued and enlarged and that additional personnel be selected from various parts of the country and representative of different types of employment within the profession. The scope of the committee's work is defined as:

> "To study the present activities of the Society with respect to human relations, welfare, and public relations of the Society and of its members, and to make recommendations for changes or amplification of such activities."

The committee is to study and report to the Board on any of the Society's activities which it is believed are, or may be made, contributory to an appropriate expansion of the human and public relations of the Society and of its members. The committee appointed consists of Director Perry, New York, chairman; Director Barbour, Boston; Director McDonald, Atlanta; and Messrs. A. B. Purton, Salt Lake City; Thomas E. Stanton, Jr., Sacramento; E. O. Sweetser, St. Louis; and Ross K. Tiffany, Olympia, Wash.

Uniform Terminology of Membership Grades

The committee appointed at the July Meeting of the Board to study and report on a recommendation of the Engineers' Council for Professional Development that the major engineering societies adopt uniform terminology for membership grades, reported a careful study. After extensive discussion, final action was to request that the committee give further consideration to the problem and report again to the Board at its next meeting.

Annual Convention of 1935

It was voted to hold the 1935 Annual Convention at Los Angeles, the date to be fixed later after consultation with the officers of the Los Angeles Local Section.

Public Works Administration

The Board went on record as approving the continuance and the expansion of the operations of the Public Works Administration.

Adjournment

Necessary adjustments of the budget were made and routine administrative matters considered. The Board adjourned to meet at Society Headquarters, on January 14 and 15, 1935, just prior to the Annual Meeting of the Society.

Arthur S. Tuttle Is Nominated for Society President

SELECTION of the official nominee for President of the Society for 1935, by the method provided in the Constitution, was effected at a meeting of the Nominating Committee held in Chicago on October 1, 1934. The choice of the committee was Arthur S. Tuttle, M. Am. Soc. C.E. He is widely known not only because of his activities in and services to the Society but also because of his important engineering experience, particularly that with the Board of Estimate and Apportionment of New York City. His service on that board began in 1902, and after his retirement in 1928 as Chief Engineer, he was retained as consulting engineer. More recently he has served as New York State Engineer for the Federal Emergency Administration of Public Works.

In the Society Mr. Tuttle has given distinguished service in numberless ways. He has been Director and Vice-President and also has served a term as Treasurer. A more complete account of his accomplishments will appear in the December number.

Death of Calvin W. Rice

FOR 30 YEARS SECRETARY OF THE AMERICAN SOCIETY OF MECHANICAL ENGINEERS

BY THE DEATH of Calvin W. Rice on October 2, 1934, the American Society of Mechanical Engineers loses its Secretary and leader in society activity. The engineering profession at large is bereft of a sagacious counselor who gave deep study and wise judgment to many professional matters of wide import to the engineering world.

Although trained as an electrical engineer, Dr. Rice after fifteen years of industrial experience found his way into professional organization work. For almost thirty years he gave of himself without stint to his engrossing duties as Secretary of the American Society of Mechanical Engineers. During this time that society grew in importance and in the outreach of its activities. In all its efforts he was a leader and frequently a sponsor. It owes much to his wise guidance.

Inter-society affairs were always close to the heart of Dr. Rice. He believed in society collaboration and was the instigator of many joint endeavors, notably in connection with the establishment of the Engineering Societies Library and the association of the four Founder Societies in the Engineering Societies Building. Many members of the American Society of Civil Engineers have been brought in contact with him in these endeavors. Particularly the Headquarters staff will miss his sage advice and his genial personality. Engineers at large will join very generally in their sympathy with the American Society of Mechanical Engineers in the death of Dr. Rice.

Nominations Completed for 1935

THE COMPLETE roster of official nominees for Society offices for the coming year can now be announced. As noted in another column, the Society's Nominating Committee chose a candidate for President at its meeting on October 1. Candidates for the remaining offices were determined by canvass of the Second Ballot for official nominees for Vice-Presidents and Directors, which ballot was counted on October 15. Report of this ballot will be found in the November PROCEEDINGS, as required by the Constitution. The complete list thus determined is as follows:

For President:

Arthur S. Tuttle, of New York, N.Y.

For Vice-Presidents:

Zone II, D. H. Sawyer, of Washington, D.C.
Zone III, Henry E. Riggs, of Ann Arbor, Mich.

For Director:

District 3, C. Arthur Poole, of Rochester, N.Y.
District 5, Herman Stabler, of Washington, D.C.
District 7, James L. Ferebee, of Madison, Wis.
District 8, Charles B. Burdick, of Chicago, Ill.
District 9, H. S. Morse, of Indianapolis, Ind.
District 12, Ivan C. Crawford, of Moscow, Idaho
District 16, Theodore A. Leisen, of Omaha, Nebr.

These names will go to all Corporate Members of the Society in good standing as a final ballot, which is distributed 40 days before the Annual Meeting and is returnable before January 9, 1935, that is, one week before the meeting. On this final ballot provision is made for votes on other names which may be inserted at the instance of any individual member. The entire final ballot is canvassed on January 9, and the tellers' report is made on the morning of the Annual Meeting. Installation of the new President takes place immediately and the new Board of Direction usually holds its first meeting on the following morning.

The entire procedure of electing officers to the Society is characterized by dignity and lack of haste. In the minds of those who framed these provisions, there was evidently the purpose of ensuring a democratic government and the eventual election of those who are eminently qualified to represent the Society officially and to give it a capable and efficient management.

New Manual on Erection of Steel Buildings

CONTINUING A YEAR of unusual activity in the production of Manuals of Engineering Practice, Manual No. 9, on the "Erection of Steel Buildings," will soon be in the mails. This is the work of the committee of the Construction Division of the Society, on Construction Plant and Methods, through its Subcommittee on Steel Buildings.

The directions for erecting derricks for laying out structural steel work and arranging construction towers, contained in this

HOISTING A STEEL COLUMN DURING CONSTRUCTION OF THE EMPIRE STATE BUILDING (1930–1931) NEW YORK, N.Y.

Manual, should prove a useful guide for contractors' engineers. It is especially designed to serve the young job superintendent and the resident engineer, but the seasoned construction man should also find in it numerous suggestions that he could use to advantage.

No effort is made to establish standards of practice, but merely to summarize accepted good practice. Among the subjects treated are: how to erect a guy derrick; how to move a guy derrick; guy wires; handling of structural steel; personnel requirements; scaffolds and derricks; list of required equipment; and layout of construction plant.

In view of its wide scope and general interest, Manual No. 9 is being sent to all Society members in good standing, of whatever grade.

Services of Engineering Societies Library Available

THERE IS a large group of engineers who cannot visit the Engineering Societies Library because of distance or lack of time. Therefore the Library maintains a Service Bureau to make available to each engineer the information he needs. The services of the bureau range from suggesting a book or one or two articles on a given subject to the preparation of exhaustive bibliographies for use in engineering projects or in patent litigation. A loan

collection of modern American engineering books is also maintained for the convenience of members of the Founder and contributing societies.

Where bibliographies are furnished by the Service Bureau the mere possession of the list of books and articles may not solve the engineer's problem. He probably will need the information to be found in the literature. Thus it has been necessary to extend the work of the Service Bureau so as to offer approximately the equivalent of a visit to the library. When bibliographies are compiled a note is appended to each article explaining what it is about, so that the recipient of the bibliography can tell whether he needs to read a given article in whole, in part, or not at all. When he has determined what articles he should read, if he cannot obtain them at a local library, he may send a list of them to the Engineering Societies Library to have photostatic copies made. The library is equipped to supply such copies of all material on file. But even the possession of an exact copy of an article may be insufficient if the article is in a foreign language. In such cases the Service Bureau supplies exact, technically accurate transcriptions or translations.

It is to be noted that the Service Bureau has on file more than 4,000 bibliographies already made up on special engineering subjects. Any of these may be procured by paying a moderate copying charge. As an example of the type of material contained in these lists, some of the bibliographies on various aspects of bridge engineering are shown in the tabulation which follows.

Some of the other civil engineering subjects covered or partially covered by bibliographies already made up and on file are: airports, appraisal, aqueducts, arches, arenas, assessments, breakwaters, buildings, cableways, caissons, canals, cement, cofferdams, columns, concrete and reinforced concrete, construction, dams, depreciation, dikes, docks, drainage, earthwork, electrolysis, excavation, failures, fire tests, floors, flow of water, foundations, grain elevators, gravel, grouting, incineration, irrigation, lighthouses, management, models, pavement, piers, piles and piling, ports, public utilities, reservoirs, retaining walls, river regulation, roads, sea walls, sewerage, shore protection, silting, siphons, stadiums, stresses in structures, surveying, tanks, terminals, towers, traffic, tunnels, underpinning, valuation, viaducts, vibrations, walls, water supply, waterproofing, wells, zoning.

Any of these bibliographies may be brought up to date or extended as far back as may be required. Estimates for such work will be furnished by the library on request. Each reference is abstracted in the general form used by the Engineering Index and published each month in CIVIL ENGINEERING under the head, "Current Periodical Literature."

The resources of the Engineering Societies Library are thus made available to engineers everywhere. All services are rendered at cost. For information communicate with the Engineering Societies Library, 29 West 39th Street, New York, N.Y.

SEARCH NUMBER	COPYING CHARGE	NO. OF REFERENCES	TITLE OR SUBJECT OF LIST	DATES COVERED
309	$2.00	42	History of bridge engineering	1811–1906
3439A	3.00	94	Books on bridge engineering	1895–1921
3348	0.75	17	Grouting bridge pier foundations	1913–1919
323	4.00	73	Bridge failures	1839–1907
329	9.00	153	Specifications and live loads for steel highway and electric railway bridges	1875–1907
4425	1.00	12	Failure of steel bridges	1875–1924
914	5.00	75	Grade of urban bridges	1890–1915
3947	2.50	43	Railroad bridges	1907–1919
3922	2.00	29	Railroad bridge designers	1910–1923
310	1.00	15	Esthetics of bridge design	1896–1904
2993	1.50	20	Encasement of steel viaducts in concrete	1904–1919
3730	0.50	5	Bridge abutments in soft ground	1915–1921
2816	1.00	16	Concrete-filled steel shell bridge piers	1893–1915
4683	1.25	19	Bridge piers	1923–1931
3081	0.50	6	Special steels for long bridge spans	1915–1918
4051	3.00	65	Single span truss bridges	1877–1919
375	0.75	17	Cantilever bridges	1884–1907
4266	5.00	84	Suspension bridge cables	1868–1926
3395	0.50	6	Design of suspension bridges	1913–1920
525	1.25	25	Reinforced concrete arch bridges prior to 1905	1891–1904
3180	11.00	161	Development of the trunnion bascule bridge	1870–1920
2961	4.50	71	Descriptions of trunnion bascule bridges	1895–1919
4088	1.00	10	Hand-operated drawbridges	1874–1923
4674	0.75	13	Long-span vertical lift bridges	1921–1932
4719	4.50	77	Toll bridges	1907–1933

Ambrose Swasey Honored by the Engineering Foundation

IT WAS once said by Ambrose Swasey, Hon. M. Am. Soc. C.E., that "Research is the greatest force for progress in all the world—this urge to find out why." Twenty years ago Mr. Swasey made the first contribution to The Engineering Foundation for "the furtherance of research in science and engineering and the advancement in any other manner of the profession of engineering and the good of mankind."

To honor its founder on the twentieth anniversary of its establishment, The Engineering Foundation tendered Mr. Swasey a reception and dinner at The Union League Club in New York, N.Y., on October 18, 1934. By their presence on that occasion seventy representatives of the four founder societies and organizations of scientists expressed their appreciation of the work Mr. Swasey thus began two decades ago. The keynote of the addresses made at the dinner was the necessity for supporting the work of The Engineering Foundation and similar organizations dedicated to human welfare. The high spot of the evening was the presentation to Mr. Swasey of a beautifully bound testimonial, engrossed on vellum and signed by those present. It read as follows:

"To Ambrose Swasey, Founder:

"On this happy occasion, officers of the American Society of Civil Engineers, American Institute of Mining and Metallurgical Engineers, American Society of Mechanical Engineers, American Institute of Electrical Engineers, and of kindred technical societies and associations of scientists, together with present and former members of United Engineering Trustees, Inc., and of The Engineering Foundation, and other friends, have gathered to celebrate the twentieth anniversary of The Engineering Foundation and to honor you as the one whose foresight and generosity brought about in 1914 the establishment of that unique and beneficent institution.

"To you the fraternities both of engineers and of scientists express high esteem as one of their most beloved members and generous benefactors.

"In the twenty years since you made your first gift, there has been a steady growth in appreciation of your great conception of an institution through which the cooperative activities of engineers, scientists, and industrialists could contribute by research and in other ways to the advancement of the profession of engineering and the good of mankind. Great have been its contributions; they mark but the beginning of far greater service in the future.

"For your eighty-eighth birthday two months hence, we wish you health and happiness and assure you of our gratitude and abiding affection. May many succeeding birthdays bring you the satisfaction of witnessing the accelerated growth and increasing usefulness of your great undertaking, The Engineering Foundation!"

Mr. Swasey has increased his initial gift to The Engineering Foundation from time to time so that his contributions now total

three-quarters of a million dollars. Other endowments have enriched this amount, and in the period of The Engineering Foundation's activity expenditures from the income have totaled nearly $450,000. To this must be added the direct contribution of money, services, and material by cooperating organizations and individuals in the projects in which The Engineering Foundation has participated, so that expenditures upon its enterprises to date may be placed at over two and a half million dollars.

One of the first activities of The Engineering Foundation was to finance the establishment of the National Research Council in 1916, with which it has since cooperated in a number of research projects, including one on marine piling and the organization of an Advisory Board on Highway Research. In the field of civil engineering, The Engineering Foundation is perhaps best known for the aid that it has given to the recently completed ten-year study of arch dams, for its collaboration in the studies of concrete arches and steel columns for bridges and buildings, and for its aid in the researches on earths and foundations, conducted by the Society.

An inventor of renown and maker of astronomical instruments, Mr. Swasey was one of the founders of the American Society of Mechanical Engineers and one of its early presidents. In 1921 he was elected an Honorary Member of the American Society of Civil Engineers.

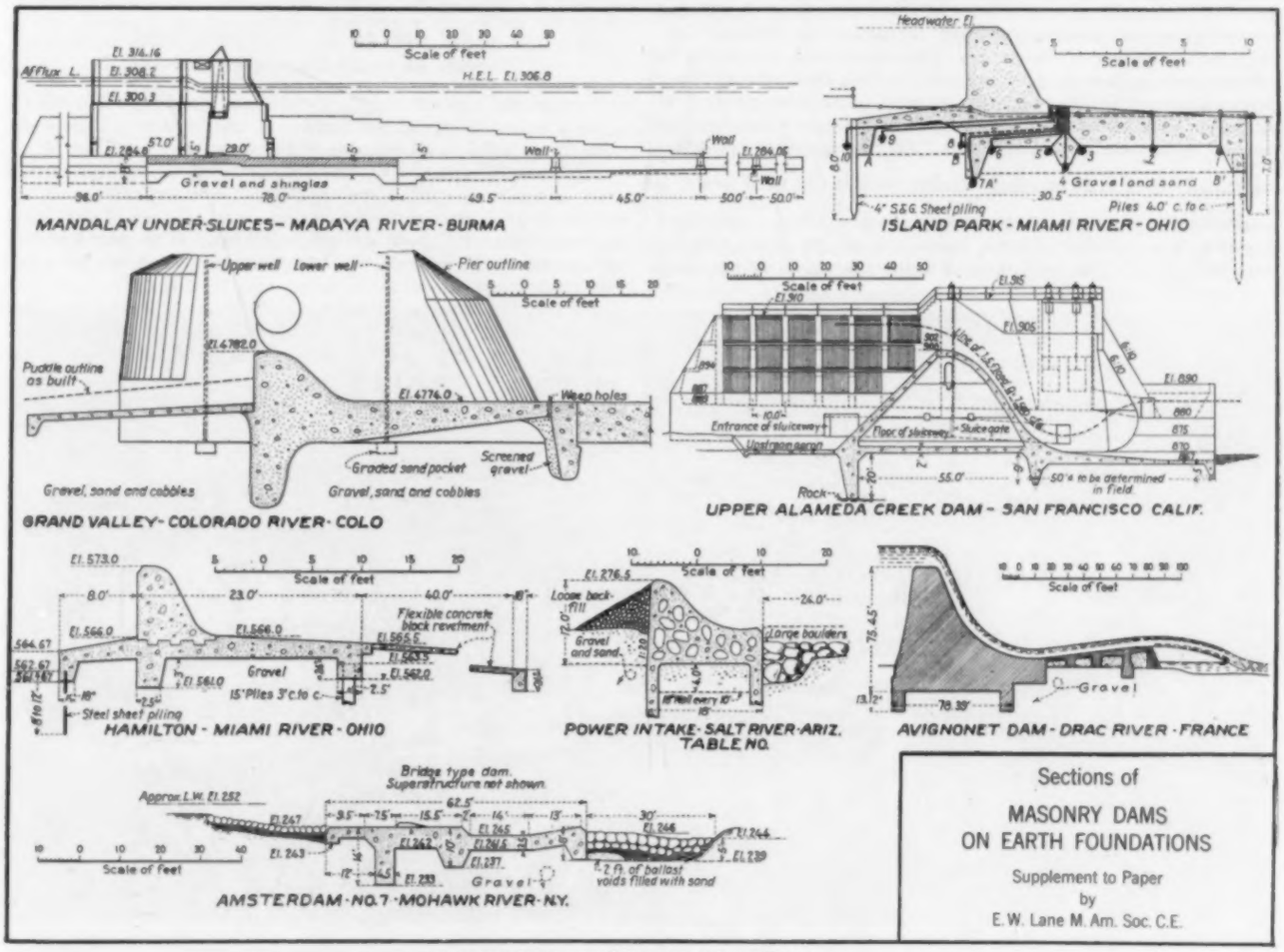
Drawings of Dams Available

IN THE September 1934 number of PROCEEDINGS appears an article by E. W. Lane, M. Am. Soc. C.E., entitled "Security from Under-Seepage—Masonry Dams on Earth Foundations." In this paper reference is made to the original manuscript, which includes drawings showing dimensions and typical sections of all the dams mentioned as well as a comprehensive bibliography. This original manuscript has been filed for reference in the Engineering Societies Library, 29 West 39th Street, New York, N.Y., for the use of those who may wish to study the underlying data in greater detail.

In addition, the Society has prepared sets of the drawings mentioned and the bibliography, with a brief index, comprising 40-page photo print leaflets in the standard 6 by 9-in. size. These may all be secured at the nominal cost of 50 cents per copy from the Secretary of the Society. The booklets form an important supplement to Mr. Lane's paper by giving a wealth of details which could only be generalized or summarized in the necessarily condensed form in PROCEEDINGS. The drawings themselves are direct reproductions of the original tracings and should prove invaluable to all engineers interested in the design or construction of masonry dams.

The index of 166 items permits convenient cross reference, not only with the dams in the supplement, but with the tables in the PROCEEDINGS paper. For many months there has been an unusual demand for this information from widely separated points. Those for whom it has special interest would do well to submit their orders early.

In this connection, perhaps, it may not be amiss to remind readers of PROCEEDINGS that a limited stock of similar photo-lithographic pamphlets, which supplement the paper by C. G. Cline in the January 1934 PROCEEDINGS, is still available, also at 50 cents per copy. Supplements of this nature are produced purely as a service to members, the price being fixed at a sum calculated to pay costs of production only.


MANDALAY UNDER-SLUICES - MADAYA RIVER - BURMA
ISLAND PARK - MIAMI RIVER - OHIO
GRAND VALLEY - COLORADO RIVER - COLO
UPPER ALAMEDA CREEK DAM - SAN FRANCISCO CALIF.
HAMILTON - MIAMI RIVER - OHIO
POWER INTAKE - SALT RIVER - ARIZ. TABLE NO.
AVIGNONET DAM - DRAC RIVER - FRANCE
AMSTERDAM - NO. 7 - MOHAWK RIVER - N.Y.
Sections of
MASONRY DAMS
ON EARTH FOUNDATIONS
Supplement to Paper
by
E. W. Lane M. Am. Soc. C.E.

SAMPLE PLATE OF MASONRY DAM DETAILS

Showing 8 of the 166 Cross Sections Included in Full Sets, Reprinted and Bound as Separates

Preview of Proceedings

In addition to the usual discussions, the November issue of PROCEEDINGS *will contain two main papers. One of these will deal with the effect of secondary stresses upon the ultimate strength of bridge truss members; the other is a description of the Springwells Filtration Plant in Detroit, Mich. Thus the November number will strike a happy balance between the theoretical and the practical.*

EFFECT OF SECONDARY STRESSES UPON ULTIMATE STRENGTH

IN the paper on the "Effect of Secondary Stresses Upon Ultimate Strength," by John I. Parcel, M. Am. Soc. C.E., and Eldred B. Murer, Jun. Am. Soc. C.E., particular attention is given to the redistribution of stress that occurs when the outer fiber stresses approach the yield point. The conclusions advanced are supported by test observations made in the experimental engineering laboratory of the University of Minnesota. Tests were made to determine the actual behavior of compression members with thin walls when subjected to unusually great secondary stresses and when loaded to failure. The authors conclude that for the types of members, and for the loading conditions investigated, the ultimate strength is not affected appreciably, even when the secondary stresses are unusually great. Yet, in the case of compression members, the relative thickness of the wall is maintained at the ratio ordinarily required by the leading standard specifications.

The paper is logically divided into two rather well-defined parts, the first dealing with the broad theory of the subject, and the latter with the experimental findings. Those who wish to obtain a previous judgment as to the scope of this excellent work will find an adequate but concise summary preceding the General Conclusions at the end of the paper. The conclusions, which are no less interesting than the premises and the data that support them, are as follows: (1) The ultimate, practical, utilizable strength of a tension member is not affected by any secondary stress action within reasonable practical limits. (2) Compression members sufficiently flexible to develop $\frac{L}{r}$ failure before the yield point is reached will usually exhibit too low a value for the secondary bending to reduce the ultimate carrying capacity of the member materially. (3) The rigid type of built column, in which alone high secondary stresses are to be expected, almost invariably fails by local overstress, usually at a point at which local buckling can readily take place.

THE SPRINGWELLS FILTRATION PLANT, DETROIT, MICHIGAN

The paper on "The Springwells Filtration Plant, Detroit, Michigan," by E. A. Hardin, M. Am. Soc. C.E., describes a plant of the rapid-sand gravity type with an ultimate capacity of 300 mgd. The reservoir is in two sections, each of which has a capacity of 20,000,000 gal, and there is provision for a third section to be added later. The paper describes the general design features, giving data for various parts of the plant. A discussion of the construction program is included, with a detailed tabulation of cost involved.

The water supply for the city of Detroit, Mich., is the largest in the world to be completely filtered by the rapid-sand method. In addition to the Springwells Plant, the Water Works Park Filtration Plant has a maximum capacity of 360 mgd. An artist's drawing of the completed structure is reproduced herewith. The sodded lawns in the foreground form the roof of the two reservoirs for filtered water. The building in the right foreground is the pumping plant with the various units of the power plant. The smaller buildings on the left include the filter buildings, and the settling basins can be seen in the left background.

Experience gained on the construction of this plant has contributed to the development of current knowledge of the flow in coagulation basins and to the development of higher rates of filter washing. The shunt system was conceived and developed in connection with this work, and various details were improved, such as filter under-drains of perforated pipe, chemical handling equipment, and rate controllers with low-head losses.

New Index to Transactions

VOLUME 84 TO 99 INCLUSIVE, 1921–1934

DURING the current month Members in good standing will receive a valuable special publication of the Society. This is the cumulative index to all volumes of TRANSACTIONS since Vol. 83.

Each yearly issue of TRANSACTIONS contains its own index. In 1921 a separate volume was published, comprehending all the entries for the entire series up to that time. Copies of this *Index to Transactions, Vol. 1–83* are still available. This year a second cumulation has been made, which supplements the earlier index and

SPRINGWELLS STATION, DETROIT'S NEW RAPID-SAND FILTRATION PLANT FROM ARTIST'S DRAWING
Ultimate Capacity 300 mgd

brings it up to date to cover all the TRANSACTIONS since 1920 and including Vol. 99 (1934). It is this supplementary index, a volume of nearly 200 pages, which will shortly be sent out. It includes complete subject and author classifications, alphabetized separately.

Mailing of the new index will be in a manner similar to the latest volume of TRANSACTIONS; that is, paper covered copies will go in a carton with the November PROCEEDINGS under the title, *Proceedings, Part 2, Transactions Index Number*. Bound copies, whether in cloth or in half morocco, will be forwarded separately to those members who receive their TRANSACTIONS in this form.

Judging by the experience of members and libraries as well as others who have had occasion to use previous Society indexes, it seems safe to conclude that the new 1934 index will fill a real need. Normally planned for publication in 1930, it has been deferred for financial reasons. As a result the present volume will be correspondingly larger and its value the more appreciated.

Badges for Christmas Gifts

AGAIN THE CHRISTMAS season approaches, and in many families there arises the problem of what to give to the man of the house. If he is a member of the American Society of Civil Engineers, there is one bit of jewelry which will be very welcome. Every member should be proud, and most members are proud, to wear the badge of the Society. The badge for Corporate Members and Affiliates is of gold, enameled in blue. It may be mounted as an ordinary pin or as a watch charm, the price for either style being $5. The Junior badge is of blue and white enamel, and the price is $2.

Although orders for these badges may be placed at any time through the Headquarters office, it would be advisable not to wait until after the first of December if the badge is desired for a Christmas gift. The only restriction on these orders is that a member may not have more than one badge at a time. If a pin or charm has previously been purchased but has been lost, a brief statement to that effect should accompany the order.

Joint Fall Meeting of the Tennessee Valley and Mid-South Sections

INVITATIONS have been issued by the newly formed Tennessee Valley Section to all Society Members, especially those in the vicinity of Knoxville, Tenn., to attend the fall meeting of the Section, which is to be held jointly with a meeting of the Mid-South Section, November 8 to 10. The principal sessions will be held in the Hotel Andrew Johnson. Details of the meeting, now being perfected, indicate an excellent program with interest concentrated on the work being done by the Government in developing the Tennessee Valley.

The registration fee is one dollar, but this will not be assessed against the ladies or students, both of whom are cordially invited. As an aid to the committee on arrangements those contemplating attending the meeting are asked to advise James G. Allen, Assoc. M. Am. Soc., C.E., chairman, at 3620 Kingston Pike, Knoxville, Tenn.

News of Local Sections

COLORADO SECTION

The Colorado Section of the Society held its first meeting of the season at the Denver Athletic Club on September 24. There were 24 in attendance at the dinner preceding the meeting, and a large group of Juniors assembled for the meeting afterwards. Numerous business matters were discussed at this session. The meeting adjourned at 9:15, after which a social hour was enjoyed.

GEORGIA SECTION

There were 18 present at a luncheon meeting of the Georgia Section held in Atlanta on October 1. After a brief business session the speaker of the occasion, C. E. Weaver, Southeastern Representative for the Federal Director of Transportation, was introduced. Mr. Weaver gave a very informative talk on modern railroad problems.

LOS ANGELES SECTION

There were 125 present at a meeting of the Los Angeles Section held on September 12. Several interesting speakers were heard. These included A. L. Sonderegger, who spoke on "Winds and Seismic Disturbances"; Dr. Ford Carpenter whose topic was "Agriculture in Relation to Localized Climate"; and Frederick W. Taylor, who discussed the overdrainage of farm lands in Iowa and Nebraska. On October 5 a dinner meeting of the Section was held in honor of President Eddy. Following dinner and a business session, J. B. Lippincott, consulting hydraulic engineer of Los Angeles, outlined the various large engineering projects being constructed in the West by means of Government aid, with specific reference to the situation in California. The chief speaker was President Eddy, who discussed various Society matters of interest. The attendance was 151.

METROPOLITAN SECTION

The Metropolitan Section held its first meeting of the fall season on October 17 in the Engineering Societies Building, with an attendance of 450. An excellent committee report of the Section, relative to the status of engineers in TERA employment, was read. The speaker of the evening was Carlton S. Proctor, who discussed the subject, "Recent Practice in the Design and Construction of Caissons and Piers." He developed the topic of bridge foundations over the last 160 years, dwelling particularly on the latest types of large and deep structures. After a brief discussion from the floor, the meeting was adjourned and refreshments were served.

MILWAUKEE SECTION

On June 21 the Milwaukee Section held its last meeting before the summer recess. Part of this session, which was held at the City Club, was devoted to business matters, and part to a talk by H. A. Sojgren, assistant mechanical engineer of the Milwaukee Railroad. Mr. Sojgren spoke on the subject, "Streamlining Railroad Passenger Trains."

SAN DIEGO SECTION

After its usual summer vacation the San Diego Section held a dinner meeting on September 27 at the Churchill Hotel. The feature of the occasion was a talk by Robert Edwards, field manager of the American Concrete and Steel Pipe Company, who spoke on the pre-cast cylinder pipe used in the construction of the Metropolitan Aqueduct. His talk was illustrated by motion pictures showing the manufacture of this pipe. A special luncheon meeting of the Section was held on October 6 in honor of Harrison P. Eddy, President of the Society. Mr. Eddy gave an interesting talk on the activities of the Society in relation to the various Government relief projects.

ST. LOUIS SECTION

There were 30 members and guests present at a luncheon meeting of the St. Louis Section held at the Mayfair Hotel on September 24. Interesting descriptions of the Annual Convention, held in Vancouver in July, were given by Frank G. Jonah and W. W. Horner, Vice-President and Director of the Society, respectively, and A. P. Greensfelder, president of the Fruin-Colnon Contracting Company. There was also a brief business session.

Student Chapter News

AGRICULTURAL AND MECHANICAL COLLEGE OF TEXAS

During the past school year the Agricultural and Mechanical College of Texas Student Chapter held eleven meetings and two banquets, at which various prominent engineers spoke. Among these were the following: Lieut. K. S. Anderson, Corps of Engineers, U. S. Army; E. N. Noyes, Director of the Society and consulting civil engineer of Dallas, Tex.; Frank T. Sheets, consulting

engineer for the Portland Cement Company, of Chicago, Ill.; and L. T. Mathis, a representative of the National Lumber and Creosoting Company, of Houston, Tex. Several students represented the Chapter at the fall meeting of the Texas Section of the Society.

HARVARD UNIVERSITY (HARVARD ENGINEERING SOCIETY)

Illustrated lectures furnished by the Society constituted the major part of the programs of meetings of the Harvard University (Harvard Engineering Society) Student Chapter during the past year. In addition to the meetings, numerous other activities were sponsored by the Chapter. These included an outing at the Arlmont Country Club in Belmont, Mass.; an inspection trip to view the recent developments of the Metropolitan Water Supply Commission of Boston; a visit to the Watertown Arsenal of the U. S. Army; a trip to the Boston Bridge Works, where the design, assembling, and fitting of structural steel shapes were studied; and a four-day spring trip. The latter, which had as its ultimate destination New York City, included visits en route to important engineering projects. In the city the group visited the Newark Airport, the George Washington Bridge, and numerous other points of interest.

UNIVERSITY OF DELAWARE

Members of the University of Delaware Student Chapter attended several meetings of the Philadelphia Section of the Society during the past school year. The six meetings held by the Chapter during the year were addressed by outside speakers, including H. C. Berry, Professor of Materials of Construction at the University of Pennsylvania; Charles Carswell, assistant engineer of the Port of New York Authority; and John Lyle Harrington, consulting engineer of Kansas City, Mo.

UNIVERSITY OF ILLINOIS

Almost all of the 1933–1934 meetings of the University of Illinois Student Chapter were addressed by outside speakers. In all there were 18 such sessions, which attracted an attendance of 1,470. Among the outside speakers were the following: F. H. Frankland, director of the Engineering Service of the American Institute of Steel Construction; Harland Bartholomew, city planner of St. Louis, Mo.; Boris A. Bakhmeteff, of the staff of Columbia University; Henry Penn, engineer for the American Institute of Steel Construction; and M. L. Fuller, a meteorologist with the U. S. Weather Bureau. A two-day inspection trip was made to the Chicago Water Tunnel and to the plant of the American Bridge Company in Gary, Ind.

UNIVERSITY OF KANSAS

There were nine meetings of the University of Kansas Student Chapter during the past school year. These sessions, which drew a total attendance of 225, were addressed by several outside speakers. Among them were LaMotte Grover, of the Kansas State Highway Department, and C. K. Mathews, of the Burns and McDonnell Engineering Company. An engineers' banquet was one of the social events particularly enjoyed.

UNIVERSITY OF KENTUCKY

Varied and interesting programs were enjoyed at the 1933–1934 meetings of the University of Kentucky Student Chapter. In all there were 20 such sessions, with a total attendance of 804. Students, members of the faculty, and outside speakers cooperated in the presentation of papers on timely engineering topics. The list of speakers included J. S. Watkins, former chief engineer of the Kentucky State Highway Department, and Hugh Meriwether, an architect of Lexington, Ky.

UNIVERSITY OF MAINE

The annual report of the University of Maine Student Chapter states that four interesting meetings were held during the 1933–1934 school year. Two interesting speakers were heard at these sessions: H. W. Wright, formerly city engineer of Bangor, Me., and Prevost Hubbard, a chemical engineer with the Asphalt Institute, of New York, N.Y. At the annual smoker held in the fall the Chapter presented a very amusing skit.

UNIVERSITY OF MICHIGAN

The annual report of the University of Michigan Student Chapter for the past school year states that papers on some technical subject are a prerequisite for membership in the Chapter. Besides a number of technical meetings, the Chapter sponsored several social events during the year, including a banquet and the showing of different motion pictures of engineering interest. Inspection trips were made to the sewage disposal plant of Flint, Mich., and to the water purification plant of Saginaw, Mich.

UNIVERSITY OF MINNESOTA

Outside speakers addressed the four meetings of the University of Minnesota Student Chapter held during the past year. In all there were 200 in attendance at these sessions. On one occasion films of Boulder Dam and of the California earthquake area were shown. The Chapter was invited to attend all the meetings of the Northwestern Section of the Society, and several of the members availed themselves of this privilege.

UNIVERSITY OF MISSOURI

Field trips had a prominent place in the activities of the University of Missouri Student Chapter during the past year. Among the engineering projects visited were Bagnell Dam; the water works of Jefferson City, Mo.; bridge construction enterprises of the Missouri State Highway Department; and the water supply systems of Sedalia and Marshall, Mo. Several technical and social meetings were also enjoyed during the year.

UNIVERSITY OF NEBRASKA

Motion pictures of various projects of engineering interest were shown at several of the 1933–1934 meetings of the University of Nebraska Student Chapter. At other sessions talks given by students, members of the faculty, and outside speakers, were greatly enjoyed. One of the interesting events of the late spring was an Engineers' Night, which attracted a large attendance.

UNIVERSITY OF NEVADA

Outside speakers addressed several of the meetings of the University of Nevada Student Chapter held during the past school year. Among these were Edward F. Berry, Professor of Structural Engineering at Syracuse University; Robert Allen, chief engineer for the Nevada State Department of Public Works; Thomas McKaig, structural and architectural engineer of Buffalo, N.Y.; and George Devore, District Manager of the Sierra Pacific Power Company.

UNIVERSITY OF NEW HAMPSHIRE

The University of New Hampshire Student Chapter reports that during the past school year it had perfect enrolment of those eligible for membership. Meetings were held frequently throughout the year, and a number of papers were presented by students at these functions. Several special sessions, with outside speakers, were also enjoyed. Numerous inspection trips were taken to places of engineering interest in the vicinity of the university.

UNIVERSITY OF NORTH CAROLINA (WILLIAM CAIN ENGINEERING SOCIETY)

Bi-monthly meetings were held by the University of North Carolina Student Chapter during the past year. The program for several of these occasions consisted of the presentation of papers by students, and the illustrated lectures furnished by the Society also proved very popular. Perhaps the most interesting activity of the year was a three-day inspection trip, which included all features of engineering interest in and around Norfolk, Va.

UNIVERSITY OF OKLAHOMA (STADIA CLUB)

Among the outside speakers who appeared at meetings of the University of Oklahoma (Stadia Club) Student Chapter during the past school year was E. H. Parker, chief engineer of the Reinhart-Donovan Construction Company, of Oklahoma City. The high light of the school year for the engineering students was the annual St. Patrick's Day celebration and open house, at which numerous interesting exhibits were displayed. After a lapse of

several years the annual inspection trip was again enjoyed. The itinerary of this trip, which lasted a week, included Bagnell Dam in Missouri; the plants of the American Bridge Company and the Illinois Steel Company in Gary, Ind.; the water supply and sewage disposal systems of Chicago, Ill.; and building operations in St. Louis, Mo.

UNIVERSITY OF NEW MEXICO

Student papers were presented at the ten meetings of the University of New Mexico Student Chapter held during the past school year. On May 9, 1934, the Chapter acted as host to the New Mexico Section of the Society at a meeting, which was addressed by E. B. Van de Greyn, New Mexico State Highway Bridge Engineer. Included in the year's activities were three inspection trips. The engineering projects visited on these occasions were the diversion dam of the Middle Rio Grande Conservancy District, the San Acacia diversion dam, and Boulder Dam. The last was a six-day trip, which included many points of scenic as well as engineering interest.

UNIVERSITY OF PITTSBURGH

Weekly meetings were held by the University of Pittsburgh Student Chapter during the past school year. Interesting and varied programs were presented at these sessions, including talks by students, members of the faculty, and outside speakers. Among the latter were E. B. Roberts, of the personnel department of the Westinghouse Electric and Manufacturing Company; Charles M. Upham, of the American Road Builders' Association; Nathan B. Jacobs, vice-president and treasurer of Morris Knowles, Inc.; and J. M. Fitzgerald, vice-chairman of the Committee on Public Relations of the Eastern Railroad Presidents' Conference. The Chapter boasted 100 per cent enrolment of those eligible for membership.

UNIVERSITY OF TENNESSEE

Several interesting meetings were enjoyed by the University of Tennessee Student Chapter during the 1933–1934 school year. Among the outside speakers who gave talks at these sessions were Erwin Harsch and Barton Jones, engineers with the Tennessee Valley Authority, and H. H. Hale, city engineer of Knoxville, Tenn. The Society's illustrated lectures were shown at some of these meetings.

UNIVERSITY OF TEXAS

The annual report of the University of Texas Student Chapter states that the year 1933–1934 was unusually successful. Although papers were presented by students and members of the faculty, outside speakers were also heard at every meeting. Among these were W. E. Maxwell, Plant Manager of the National Creosote Company; T. B. Warden, District Engineer for the Portland Cement Association; E. N. Noyes, Director of the Society and consulting civil engineer of Dallas, Tex.; and C. M. Davis, president of the Texas Section of the Society. Several inspection trips were made during the year. The points of engineering interest visited included a reinforced concrete bridge spanning the Brazos River at Waco, Tex.; the water filtration plants at Fort Worth and Dallas; the Tips Engine Works at Austin, Tex.; and the Republic Portland Cement Company at Uvalde.

UNIVERSITY OF UTAH

Illustrated lectures loaned by the Society formed the nucleus of entertainment at the twelve meetings held by the University of Utah Student Chapter during the past school year. In March a four-day "Engineers' Round-up" was sponsored by the Chapter, whose exhibit won second place in competition with those of all the other engineering societies at the university. The final event of the season was a seven-day inspection trip, which included such points of interest as Boulder Dam, the engineering projects of interest in southern California, and the San Francisco–Oakland Bay Bridge.

UNIVERSITY OF VIRGINIA

Besides holding several technical meetings, the University of Virginia Student Chapter participated in various other activities of an engineering nature during the past school year. These included an inspection trip to the Virginia Bridge and Iron Company at Roanoke and participation in the annual engineers' banquet. The Chapter has also been instrumental in the formation of an Engineering Council to coordinate the activities of the professional societies.

UNIVERSITY OF WASHINGTON

In addition to several technical meetings, the University of Washington Student Chapter enjoyed numerous field trips during the past year. These trips included the inspection of the following plants: the Pacific Coast Steel Corporation, the Wallace Bridge and Structural Company, and the Washington Wood Preserving Company. The Chapter took an active part in the biennial engineers' open house held every two years.

UNIVERSITY OF WISCONSIN

Students and members of the faculty addressed all the meetings of the University of Wisconsin Student Chapter during the past year. On May 10 Henry E. Riggs, Director of the Society, spoke before a special dinner meeting on the subject of Society activities. Among other social events the Chapter prepared a float for the annual St. Patrick's Day parade.

UNIVERSITY OF WYOMING

The annual report of the University of Wyoming Student Chapter states that seven meetings were held during the past school year and that the total attendance at these sessions was 153. The list of outside speakers included E. C. Barritt, Wyoming State Engineer; N. Swanson, a local contractor; and F. D. Bellamy, Field Engineer for the U. S. Coast and Geodetic Survey on local survey work.

VANDERBILT UNIVERSITY (ROBERT H. MCNEILLY)

Illustrated lectures loaned by the Society formed the nucleus of entertainment at 1933–1934 meetings of the Robert H. McNeilly (Vanderbilt University) Student Chapter. In addition, several outside speakers were also heard. These included L. O. Hopkins, an engineer with the Nashville Bridge Company; James Fitch, of the Government planning board for slum eradication; and T. Alfred Fleming, of the National Board of Fire Underwriters. Some of the meetings were devoted to the presentation of papers by members of the Chapter. Several smokers were held during the year and proved the favorite social diversion.

VILLANOVA COLLEGE

The Villanova College Student Chapter reports a 100 per cent enrolment of those eligible for membership during the past year. Talks given by members of the Chapter and the presentation of illustrated lectures furnished by the Society were the chief features of entertainment at the various meetings. Motion picture films loaned by commercial organizations also proved of interest, as did several inspection trips. The latter included such points of interest as the Philadelphia Navy Yard, the Budd Wheel Company, and the Brill Car Company.

VIRGINIA MILITARY INSTITUTE

During the past school year the Virginia Military Institute Student Chapter enrolled all of those eligible for membership. The presentation of papers by members of the Chapter was a feature of each of the 18 meetings held during the year. These papers covered a wide range of engineering topics. Among the outside speakers who addressed sessions were H. E. Willis, of the Division of Tests of the U. S. Bureau of Public Roads; Hale Sutherland, Professor of Civil Engineering at Lehigh University; L. P. Smithey, architect and engineer of Roanoke, Va.; and E. M. Hastings, chief engineer of the Richmond, Fredericksburg and Potomac Railroad Company.

VIRGINIA POLYTECHNIC INSTITUTE

Varied programs of entertainment marked the 1933–1934 meetings of the Virginia Polytechnic Institute Student Chapter. Talks were given by members of the Chapter and of the faculty, and on some occasions motion pictures were shown. The social events of the year included several smokers.

ITEMS OF INTEREST

Engineering Events in Brief

Civil Engineering for December

In one of the articles in the December issue Carlton S. Proctor, M. Am. Soc. C. E., discusses the complicated foundation problems that have arisen during the construction of the piers for the San Francisco–Oakland Bay Bridge. The caissons, which are unusual in size, depth, and design, had to meet the unprecedented condition of extending to bedrock a maximum distance of 242 ft below high water. Mr. Proctor describes their unique design and the seating of these huge domed cofferdams in place.

Engineers recognize the fact that they must design attractive structures as well as practical ones. The relation of architecture to bridge engineering is the subject of an interesting article by Harry J. Engel, Jun. Am. Soc. C.E., which is also scheduled for this number. Taking a group of notable bridges, he shows how in each case both architecture and engineering have contributed to the attractiveness of the completed structure.

As part of a state-wide program for the elimination of grade crossings, the Delaware and Hudson Railroad has recently completed an interesting project at Whitehall, N.Y., which cost nearly a million and a half dollars. James M. MacMartin, M. Am. Soc. C.E., has prepared an article for the December issue explaining the necessity for the work and showing how it was accomplished.

Hydraulic Laboratory Research in Germany

By Herbert D. Vogel, Assoc. M. Am. Soc. C.E.
First Lieutenant, Corps of Engineers U. S. Army; Command and General Staff School, Fort Leavenworth, Kans.

During the summer of 1934 Lieutenant Vogel visited Germany where he gave special attention to its many hydraulic laboratories. In this article he presents his observations as to the present activities of these laboratories. He is well fitted to make such a report, for he spent the school year 1928–1929 studying hydraulic laboratory practice at the Technical University of Berlin, where he received the degree of doctor of engineering. After a term of duty as Director of the U. S. Waterways Experiment Station, at Vicksburg, Miss., Lieutenant Vogel was recently assigned to his present post.

In the Germany of today more stress is being laid on the conduct of practical experiments in the field of hydraulics than recently. The result is that less time is being given to pure research, although investigations of a theoretical nature are generally undertaken when a definite end, bearing on some practical problem, is in sight. Research, however, of the kind that is conducted for theoretical reasons alone, or only because something might come of it, has been abandoned in most cases, for the time at least. It is probable that this condition is only temporary, arising from the same economic situation that holds the rest of the world in its grasp, but of course no one can safely predict the future. One can only say definitely that, at the present time, little work, except that performed by students as requirements for degrees, is being carried on in the river laboratories of the technical universities. In Berlin, Munich, and Dresden the story is the same, and at the latter institution only $140 was available for the hydraulic laboratory during the past fiscal year. Similarly, at Danzig only student exercises are being performed for the most part. These include routine experiments relative to the flow of water over weirs, the effects of river contractions, the design of lock chambers, and the movement of bed materials in a river.

Probably the most active river laboratory of Europe is the Preussische Versuchsanstalt für Wasserbau, Schiffbau, und Erdbau, located on the Lock Island of the Spree Canal in Berlin. Here more work is being carried out than ever before, but as might be expected, by far the greater part of it is practical in nature. On the theoretical side, however, a series of experiments is being conducted to determine with greater finality the manner in which river beds are developed in nature. This includes investigations of bed load movement in a tilting flume and an experiment dealing with the movement of sedimentary materials in the bend of a stream. Nothing tangible has yet come of the latter study, and nothing has been reported on it. On the other hand, the investigations relative to the subject of bed load have been very fruitful. They now appear to be nearing completion and to give promise of making available many valuable data in the near future. Specifically, these studies of bed load include the following: (1) investigation of the movements of materials of varying mixtures through the employment of different colored sands; (2) investigation of the movement of materials of different specific gravities, including barium sulfate, sand, and lignite; and (3) a supplemental investigation of granulated amber as a bed material.

Model of the Yellow River at the Walchensee Laboratory, Bavaria

The practical experiments that have been made in this laboratory within the past year include: a study of the Elbe in the vicinity of Hamburg, where the ebb and flow of the tides must be considered, and where bed movement is a matter of importance; and experiments for the Adolf Hitler Canal in Upper Silesia. Studies to determine the design of dams and the best forms of protective works for the control of erosion below aprons are being made for proposed improvements in Turawa and Upper Silesia. An experiment for the design of the Saale Locks has also been proposed.

An investigation to determine the effects of cut-offs in the Durme River within the region of tidal influence is now well on the way to completion, and it is interesting to note that this experiment is being conducted for and at the request of the Belgian Government by the Berlin engineers. At Marquardt, the site of the outdoor experimental grounds of the institute, a large model of the Elbe River, in the vicinity of Schoenebech, is being constructed to an undistorted scale of 1:50. Performance tests were begun in the early fall of this year.

Altogether, 15 engineers and 18 laborers are employed in the hydraulics division of the laboratory. Funds expended by all three divisions—hydraulics, marine design, and soils—last year aggregated about $140,000.

In recent years there have been few new developments in either the marine or soils

EIGHTY-SECOND ANNUAL MEETING, January 16–19, 1935, in New York, N.Y., at Society Headquarters

divisions or, in fact, even in the hydraulics division, which because of its wide scope should show the greatest and most diversified development. The field of greatest progress, so far as can be determined by casual observation, is in the apparatus employed for measuring stresses on model ships in model locks. Automatic chronographic recorders have been developed to such a remarkable degree of perfection that one operator, putting the model through its paces, can obtain a complete record of performance in a few minutes.

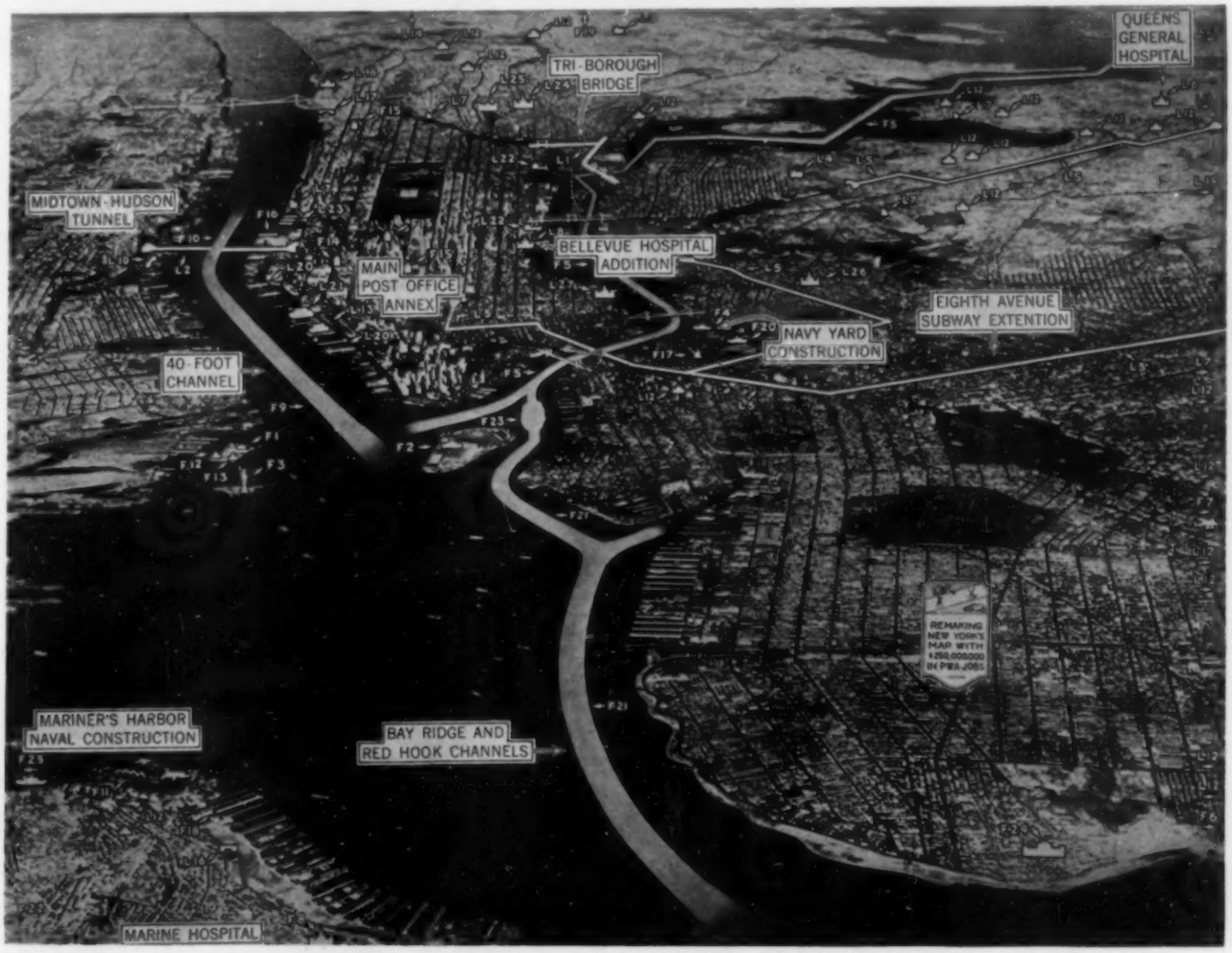
Important work on the movement of suspended materials by rivers is being conducted at Walchensee, in Bavaria, by the great pioneer of modern river laboratory practice, Dr.-Ing. Hubert Engels. Here the problem under consideration is the Yellow River of China, for which Professor Engels has been retained by the Chinese Government to work out a plan of regulation. He has already conducted experiments to determine the characteristics of natural soil materials in suspension, and to find a material which will adequately simulate those in the model now under construction. As a result of the preliminary tests, powdered coal has been found to give satisfactory results, and its future use is being planned. The model with scales of 1:165 and 1:82.5, will be 150 m long and 9 m wide when completed. Its useful length will be 120 m. and its slope 0.0012. Although this model is not an exact reproduction of the Huang Ho, or Yellow River, it is expected that general principles will be determined that can be applied to this river throughout its entire length.

In Hamburg, at the Hamburgische Schiffbau-Versuchsanstalt, a special flume was installed May 1934, for testing ships against the action of flowing water. The experiments subsequently conducted in this flume are the first of their kind ever attempted on a scale large enough to assure satisfactory results. Propellers serve to circulate water up to 8,000 liters per sec through the channel, which is 95 m long, $1/3$ m deep, and 3 m wide. Another recent and valuable installation is a "high-speed" tank, in which models may be towed through still water at a speed of 25 m per sec. The institute employs 40 engineers and 50 laborers to carry out a wide variety of experiments, ranging from tests of canal boats to investigations of ocean liner and speedboat designs.

Public Works Administration Projects in New York City Area

AN UNUSUAL composite aerial photograph of part of the metropolitan area of New York is here printed from one section of a negative made at a height of 26,000 ft by Captain Albert W. Stevens, Air Corps, U. S. Army. Those who attended the Annual Meeting in New York in January 1934, will remember Captain Stevens for his fascinating and instructive illustrated address on "Aerial Photography." More recently, he was one of three officers who made an exploration of the stratosphere in a sealed gondola attached to the huge balloon which took off from a location in the Black Hills of South Dakota and landed safely in Nebraska after the balloon ripped at an elevation of $11\frac{1}{2}$ miles.

On the photograph here reproduced have



LOCATING THE PUBLIC WORKS PROJECTS IN METROPOLITAN NEW YORK

been superimposed symbols representing a part of New York's PWA projects, which involve an expenditure totaling a quarter of a billion dollars. Projects sponsored by New York or nearby communities are designated by "L" for local, whereas those undertaken by the Federal Government are indicated by "F." The nomenclature used for identification of the projects and their estimated costs follow:

L-1	Tri-Borough Bridge	$44,200,000
L-2	Midtown-Hudson Tunnel	37,500,000
L-3	Bronx "Hillside" Housing Project	5,060,000
L-4	Woodside, Queens, "Boulevard Gardens" Housing Project	3,450,000
L-5	Eighth Avenue Subway extension	23,160,000
L-6	Queens General Hospital	800,000
L-7	Garbage disposal plants	4,000,000
L-8	Bellevue Hospital additions	4,978,000
L-9	Newark Airport	22,000
L-10	Hoboken Viaduct	136,000
L-11	Bayside High School	2,250,000
L-12	Grade and high schools	2,268,500
L-13	Pier improvements	567,500
L-14	Yonkers sewers, water mains, and pumping plant	1,074,000
L-15	Great Neck sewers	705,000
L-16	Grade crossing elimination	460,000
L-17	Railway-highway crossing	385,500
L-18	Jewish Memorial Hospital	350,000
L-19	School	384,000
L-20	Concrete deck pier	744,000
L-21	Two-story shed and relocation of sewers	1,282,000
L-22	Two pier dumping platforms and sheds	307,000
L-23	Pier sheds	1,193,000
L-24	Mott Haven Health Center	185,000
L-25	Jail	989,000
L-26	Health Center	214,000
L-27	Laboratory building and new animal house	700,000
F-1	Ellis Island: new buildings, sea wall, remodeling	$1,152,000
F-2	Governor's Island: Fort Jay, buildings, water system, roads, hospital, etc.	582,000
F-3	Statue of Liberty: improvements to landing walls, interior lighting system, etc.	31,000
F-4	Construction of new naval vessels, Brooklyn Navy Yard	4,013,000
F-5	40-ft channel to Navy Yard, 35-ft channel to Long Island Sound	1,456,000
F-6	Rockaway Inlet, 12-ft channel and lights	77,000
F-7	Mitchell Field improvements	471,000
F-8	Marine Hospital	2,272,000
F-9	40-ft channel	100,000
F-10	Channel widening	656,000
F-11	Riprap protection	18,400
F-12	Quarantine station	72,000
F-13	Quarantine station	130,000
F-14	Main Post Office Annex	5,143,000
F-15	Post offices	8,214,400
F-16	Dredging	48,000
F-17	Navy Yard improvements	1,006,000
F-18	Reconditioning ships and pier improvements	387,500
F-19	Remodeling Mount Vernon Federal Building	90,000
F-20	Construction of Coast Guard cutters	3,719,000
F-21	Channel improvements	175,000
F-22	Perth Amboy anchorage area	817,000
F-23	Removing Buttermilk Channel shoal	131,000
F-24	Fort Hamilton, construction of non-commissioned officers' quarters	9,300
F-25	Construction of Navy Vessels, Mariner's Harbor	10,050,000

NEWS OF ENGINEERS

From Correspondence and Society Files

MILO F. OHR was recently appointed State Engineer Inspector for the state of Michigan under the Federal Emergency Administration of Public Works.

CLAUDE A. J. ENGLISH has been appointed an engineering draftsman with the U. S. Biological Survey, Department of Agriculture, with headquarters in Washington, D.C.

T. P. PEARSON is now superintendent of the Standard Dredging Company of Myrtle Beach, S.C.

WARREN J. MEAD has resigned as Professor of Geology at the University of Wisconsin to accept an appointment to the Department of Geology of Massachusetts Institute of Technology.

WALDEMAR J. LANDWEHR has accepted a connection with the Madison Metropolitan Sewerage District, of Madison, Wis. Prior to that he was employed by Daniel W. Mead and Charles V. Seastone, engineers of the same city.

JOHN DE JONG was recently assigned as a Reserve Officer on duty with the CCC, Company 990, Camp SP12, Big Sur, Calif.

Changes in Membership Grades

Additions, Transfers, Reinstatements, Deaths, and Resignations

From September 10 to October 9, 1934, Inclusive

ADDITIONS TO MEMBERSHIP

BROWN, SHELDON ABBOTT (Jun. '34), 3963 Wolff St., Denver, Colo.

CALVERT, JOHN THORNTON (Jun. '34), with John Taylor & Sons, Caxton House, Westminster, S. W. 1 (Res. 61, Half Moon Lane, Herne Hill), London, S. E. 24, England.

CAROTHERS, HENRY PORTER (Jun. '34), Office Engr., State Highway Dept., 7th Floor, Highway Bldg., Austin, Tex.

CROCKER, FORREST SAMUEL (Jun. '34), 1231 First National Bank Bldg., Denver, Colo.

DOVERSPIKE, HARRY CLYDE (Assoc. M. '34), Asst. Civ. Engr., U.S. Engrs., St. Paul (Res., Y. M. C. A., Winona), Minn.

GETCHELL, WENDELL BRUNGART (Jun. '34), Draftsman-Checker, State Dept. of Highways (Res., 205 Prospect St.), Uniontown, Pa.

HARTMAN, PAUL (Jun. '33), 609 West 114th St., New York, N.Y.

HELLSTROM, BO MANNE (M. '34), Cons. Civ. Engr. (Vattenbyggnadsbyran), Care, Rendel, Palmer & Tritton, 55 Broadway, London, S. W. 1, England.

JUDGE, WILLIAM JOSEPH (Assoc. M. '34), Constr. Mgr., John Monks & Sons—Ulen & Co., Serres, Greece.

LAMOTTE, FRANK WILLIAM (Assoc. M. '34), Engr., 1818 Henderson St., Columbia, S.C.

MCKAY, DONALD MIDDLETON (Jun. '34), Bill of Materials Clerk, Eng. Dept., The Pfaudler Co. (Res., 206 Rutgers St.), Rochester, N.Y.

MYERS, MASON KAY (Jun. '34), Junior Topographic Engr., U. S. Geological Survey, 215 East Main St., Visalia, Calif.

PEEL, HARRY HERBERT (Assoc. M. '34), Office Engr., State Highway Dept., 503 Culbertson St., Paris, Tex.

RATH, AUGUST FREDERIC (Assoc. M. '34), Res. Engr., State Highway Dept., Box 1493, Globe, Ariz.

RUSSELL, TOM (Assoc. M. '34), Asst. City Engr., (Res., 2212 South Palmetto St.), Sioux City, Iowa.

SHARP, WILLIAM HOWARD (M. '34), Engr., Highway Control, Bureau of Maintenance, State Highway Dept., 216 Preston St., Windsor, Conn.

SOLER-LÓPEZ, ERNESTO ANTONIO (Jun. '34), Insp., War Dept., U. S. Engr. Office, San Juan, Puerto Rico.

TAWSE, BERTRAM WILLIAM (Assoc. M. '34), Asst. Res. Engr., Sir Alexander Gibb & Partners, Galloway Water Power Development; Royal Hotel, Kirkcudbright, Scotland.

VANCE, JAMES DAVIS (Jun. '34), Chf. of Party, U. S. Coast and Geodetic Survey; 308 South High St., Harrisonburg, Va.

MEMBERSHIP TRANSFERS

RIDGWAY, ROBERT (Past-President) (Jun. '88; M. '03; Hon. M. '34).

KETCHUM, MILO SMITH (Assoc. M. '01; M. '08; Hon. M. '34).

LIVESAY, DURWARD PAUL (Jun. '30; Assoc. M. '34), Bridge Designer, State Highway Dept., 221 East 11th St., Olympia, Wash.

MIMS, STALEY WOOD (Jun. '32; Assoc. M. '34), Res. Engr., State Highway Div., Bronson, Tex.

MOHR, ERIK (Jun. '28; Assoc. M. '34), Sdr. Fasanvej 65, Copenhagen, Denmark.

REINSTATEMENTS

CHAMBERLIN, STEPHEN JOHNES, Jun., reinstated Sept. 10, 1934.

DEFABRITIS, LAURENCE LAMONT, Jun., reinstated Sept. 27, 1934.

EATON, HAROLD IRVING, M., reinstated Sept. 19, 1934.

FREERET, WILLIAM ALFRED, Assoc. M., reinstated Sept. 13, 1934.

KELSON, MIHILLS OLIE COURTNEY, Jun., reinstated Oct. 8, 1934.

LEFEBER, ALFRED, Assoc. M., reinstated Sept. 13, 1934.

LUSH, WILLIAM HENRY, Assoc. M., reinstated Sept. 13, 1934.

MASONI, JOHN GABRIEL, Jun., reinstated Sept. 10, 1934.

MILLER, JOSEPH JAMES, Assoc. M., reinstated Oct. 2, 1934.

TOTAL MEMBERSHIP AS OF OCTOBER 9, 1934

Members	5,701
Associate Members	6,073
Corporate Members	11,774
Honorary Members	19
Juniors	2,863
Affiliates	98
Fellows	4
Total	14,758

NEWCOMER, ALBERT WALDO, Assoc. M., reinstated Oct. 4, 1934.

OWENS, TERRENCE JOSEPH, Jun., reinstated Sept. 7, 1934.

PETERSEN, LUDWIG, Assoc. M., reinstated Oct. 4, 1934.

SADLOW, JOSEPH FRANK, Affiliate, reinstated Oct. 2, 1934.

SPENCER, FREDERICK BURR, M., reinstated Sept. 21, 1934.

STURGEON, RALPH ANDREW, Assoc. M., reinstated Sept. 7, 1934.

SUMMERS, GEORGE JOSLYN, Assoc. M., reinstated Sept. 19, 1934.

WALTON, HOWARD LACY, Assoc. M., reinstated Sept. 25, 1934.

WOOD, ARTHUR WILLIAM, JR., Jun., reinstated Sept. 10, 1934.

RESIGNATIONS

SABATELLI, EMIL ALBERT, Jun., resigned Sept. 20, 1934.

WILLIAMS, ROGER BUTLER, JR., Assoc. M., resigned, Oct. 8, 1934.

DEATHS

ALLEN, JAMES PIERSON. Elected Jun., Mar. 5, 1879; M., June 4, 1884; died Sept. 11, 1934.

BOWIE, ALFRED WILLIAM. Elected M., June 11, 1917; died Sept. 9, 1934.

DAVIDSON, WILLIAM CLARENCE. Elected Assoc. M., Dec. 6, 1915; M., June 2, 1920; died Sept. 1, 1934.

DUNLAP, HERBERT ALLEN. Elected Assoc. M., June 1, 1920; died Sept. 9, 1934.

ERICSON, ERIC GUSTAF. Elected M., Dec. 31, 1913; died Sept. 22, 1934.

GRADY, JOHN FRANCIS. Elected Assoc. M., Oct. 26, 1931; died Aug. 4, 1934.

LANG, OLIVER HOWARD. Elected M., Jan. 17, 1927; died Aug. 24, 1934.

MURPHY, EDWARD CHARLES. Elected Assoc. M., Sept. 5, 1900; M., March 31, 1908; died Sept. 18, 1934.

PENNIMAN, WILLIAM MERIT. Elected M., May 28, 1923; date of death unknown.

SLATER, JOSEPH MANSFIELD. Elected M., Sept. 3, 1912; died Sept. 14, 1934.

STEWART, FRANCIS ALEXANDER. Elected Jun., July 16, 1928; Assoc. M., Oct. 24, 1932; died Sept. 8, 1934.

Men Available

These items are from information furnished by the Engineering Societies Employment Service, with offices in Chicago, New York, and San Francisco. The Service is available to all members of the contributing societies. A complete statement of the procedure, the location of offices, and the fee is to be found on page 85 of the 1934 Year Book of the Society. To expedite publication, notices should be sent direct to the Employment Service, 31 West 39th Street, New York, N.Y. Employers should address replies to the key number, care of the New York Office, unless the word Chicago or San Francisco follows the key number, when the reply should be sent to the office designated.

CONSTRUCTION

GRADUATE CIVIL ENGINEER; Jun. Am. Soc. C.E.; 29; New York State license; 1 year on construction and design of highways; 1 year as estimator of subway structures; 4 years as chief of party, inspector, etc., on construction of water tunnel, shaft sinking, caisson work, tunnel driving and lining. Good designer and computer. Speaks Russian, German, and French. C-5839.

CONSTRUCTION SUPERINTENDENT; Assoc. M. Am. Soc. C.E.; 42; married; 21 years experience on construction of industrial buildings, gas, electric, and steam plants, street railways, sugar and oil refineries, municipal work, engineering supervision, and appraisal work. Location immaterial. Excellent references. C-9526.

CONSTRUCTION ENGINEER; Assoc. M. Am. Soc. C.E.; 41; good executive. Excellent health. Accurate and conscientious. Good record. Can estimate and get jobs; can design and construct roads, buildings, bridges; 3 years on housing developments; 1 year on subway work; 8 years buildings; 8 years roads, bridges, sewers. Exceptional references. D-3424.

DESIGN

STRUCTURAL ENGINEER; Assoc. M. Am. Soc. C.E.; 35; married; M.S., University of Illinois. Thorough experience in design of buildings, rigid-frame bridges, subways; concrete and structural steel. Aptitude for analytical work. Wide theoretical and practical knowledge of analysis of continuous frames. D-1507.

STRUCTURAL DESIGNER; Assoc. M. Am. Soc. C.E.; 34 years old; college graduate; married; 10 years experience with fabricators and railroads in designing, estimating, and detailing of heavy bridges, mill, and office buildings. Desires any suitable position and will consider sales-work. Location immaterial. A-3840.

CIVIL AND STRUCTURAL ENGINEER; Jun. Am. Soc. C.E.; licensed professional engineer; 30; B.S. in C.E. and C.E. degrees; experienced as a designer, draftsman, checker, detailer, and estimator of structural steel and reinforced concrete structures. References and samples of work will be sent on request. C-6186.

EXECUTIVE

CIVIL ENGINEER, COST ACCOUNTANT; Assoc. M. Am. Soc. C.E.; 51; married; graduate; 27 years in concrete building construction, design, supervision, time study, cost accounting, both in field and in charge of department. Also some structural steel drafting and valuation for utilities. Excellent references. Available now. Location immaterial. D-3178.

CONCRETE SPECIALIST; Assoc. M. Am. Soc. C.E.; 32; 10 years experience; capable of organizing and supervising concrete inspection, and concrete-testing laboratory on large projects. Experience on special investigation and reports on concrete and the behavior of its constituents. Several papers and discussions published in technical periodicals. Excellent references. C-8022.

ENGINEER-OPERATOR; M. Am. Soc. C.E.; experience in utilities field; transportation specialist with extensive administrative and consulting experience; technical graduate, New York license. Desires opportunity, preferably in the operating department of a utility, management, or investment corporation or financial institution, where experience can be utilized. Title immaterial. Eastern headquarters preferred, but would travel. Now available. B-7775.

PATENT ATTORNEY; Assoc. M. Am. Soc. C.E.; Capable of organizing and managing patent department. University graduate. Professional engineering degree. Member of New York Bar. Registered U. S. and Canadian patent attorney. Broad business experience; 8 years patent experience with old established patent law firm and patent department of large corporation. B-1819.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; 39; married; 3 years topographic surveys, Philippine Islands; 6 years designing and directing surveys of highway construction and required structures; 6 years in construction of power plant and appurtenant structures and surveys of dam sites; 2 years as park engineer. Available immediately. Location anywhere. D-732-324-A-5 San Francisco.

SANITARY ENGINEER; Assoc. M. Am. Soc. C.E.; graduate C.E.; 12 years experience in design and construction of sewers and sewage treatment plants; in charge of a million-dollar sewerage project since its inception, financed with PWA funds; will be available in four weeks. C-874.

ENGINEER EXECUTIVE, M. Am. Soc. C.E.; registered professional engineer, Pennsylvania. Desires connection as district representative or in supervising capacity in engineering, operation, or sales of manufacturing industry. Available immediately. C-5095.

ENGINEER; M. Am. Soc. C.E.; Member A.S.M.E.; of American descent; many years experience in gold mining and in the erection and operation of industrial and chemical plants; desires position as superintendent, manager, or chief engineer. Has had 12 years foreign experience and is willing to go anywhere. B-354.

TELEPHONE ENGINEER; Jun. Am. Soc. C.E.; 29; married; 10 years experience, all phases telephone management, engineering, construction, maintenance, outside plant, manual and step-by-step dial offices; business office experience. Desires position with telephone company requiring wide practical knowledge; now active charge of plant maintenance, 11 offices, 16,000 stations; available one months notice. D-3059.

JUNIOR

ENGINEER; Jun. Am. Soc. C.E.; 22; single; B.S. in C.E., University of Virginia; 3 months experience in sanitary survey and sanitary engineering design; 1 year topographic and general surveying; 3 months concrete construction. Desires position in sanitary engineering or other civil engineering work. Excellent references. D-2530.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 22; single; B.S. in C.E., Newark College of Engineering; 1 year as assistant in engineering drawing office; 1 year as traffic surveyor; desires opportunity and experience in any branch of civil engineering. Location and salary secondary; foreign country suitable; available immediately. D-3397.

CIVIL ENGINEERING GRADUATE, 1933; Jun. Am. Soc. C.E.; 4 months experience. Desires opportunity in any branch of engineering. Available immediately. Location immaterial. D-2683—4750 Chicago.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 27; married; graduate of Lehigh University, 1928, C.E. degree; Tau Beta Pi; 2 years as fieldman on sewer construction; 6 months as concrete draftsman; 1 year with Port of New York Authority on construction. Desires engineering or teaching position. Location immaterial. C-6787.

COLLEGE GRADUATE; C.E.; student member; desires opportunity for practical experience. Has some limited experience in surveying. No preference as to type of work. Salary and location are minor considerations. D-3412.

CIVIL ENGINEERING GRADUATE; Jun. Am. Soc. C.E.; 22; single; B.S. in C.E. (highway option), University of Illinois, 1932; Tau Beta Pi. Doing subgrade soil research. Prepared for position in any branch of civil engineering, preferably highway or structural. Location immaterial. Available. D-2043.

JUNIOR ENGINEER; Jun. Am. Soc. C.E.; 25; B.S. in C.E., 1929; desires any type of engineering teaching or construction position. Has had 2½ years experience as private tutor, as junior metallurgist, and as special mill building layout man in steel mill. D-1911.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 23; single; B.S. in C.E., in structural, hydraulic, and general C.E. options, University of Illinois, 1932. Member Tau Beta Pi; associate Sigma Xi. Summer experience on survey corps. Good draftsman. Best references. Speaks German. Location immaterial in U. S. or foreign countries. D-2504.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 23; single; graduate of Rensselaer Polytechnic Institute; 1 year experience as assistant engineer in building construction. Field knowledge of bookkeeping, estimating; 1 month on bridge construction. Good draftsman. Surveying, computations a specialty. Excellent understanding of Italian; speaks French. Willing to learn. Good references. Available now. Location immaterial. D-2423.

JUNIOR ENGINEER; Jun. Am. Soc. C.E.; 24; single; B.S. and C.E., Columbia University, 1932 and 1933; 14 months with U. S. Coast and Geodetic Survey; 1 year in hydraulic research (test flume) for thesis; engineering or teaching desired; location immaterial. D-3407.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 27; single; C.E. degree, Columbia University, 1930; 2½ years experience as assistant engineer with general contractor; design; estimating. Supervision of construction, dock and harbor work, foundations, heavy construction. Good mathematician; capable and reliable draftsman and designer. Desires engineering position of any nature. D-2682.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; B.S. in C.E.; recent graduate of Cooper Union Institute of Technology; foundation layout; experienced transitman and rodman; cost accounting of materials and labor; checking. Qualified for mechanical draftsman (Federal) and senior topographical draftsman. Good letterer; 2-year study of architecture. Salary and location open. Available. D-3416.

STRUCTURAL DESIGN; Jun. Am. Soc. C.E.; 28; married; B.S., California Institute of Technology, 1933; 6 weeks on survey of Long Beach earthquake damage; 1 year of structural design, principally of new and reconstructed school buildings of steel, wood, and reinforced concrete; indeterminate analysis. Desires civil engineering work anywhere; South American location preferred. D-3438.

ENGINEER; Jun. Am. Soc. C.E.; B.S. in C.E., Syracuse University, 1933; single; 23; 4 months graduate study on indeterminate structures; 3 months as instrumentman, U. S. Coast and Geodetic Survey work under the CWA; 2 months in charge of drainage survey; interested in construction, hydraulics, surveying. Location United States. Salary secondary. D-3439.

MISCELLANEOUS

DRAFTSMAN; Assoc. M. Am. Soc. C.E.; computer and surveyor in real estate development. Design, plan, and construction of necessary municipal improvements: roads, water, gas, electric installations, sewer and subway construction. Grade separations—railroad and highway—preliminary surveys, general planning, estimating and construction of bridges. Married: 44 years of age. Licensed professional engineer in New York. D-2092.

RECENT BOOKS

New books of interest to Civil Engineers, donated by the publishers to the Engineering Societies Library, or to the Society's Reading Room, will be found listed here. A comprehensive statement regarding the service which the Library makes available to members is to be found on page 75 of the Year Book for 1934. These notes regarding the books are taken from the books themselves, and this Society is not responsible for them.

DIE ABFLUSSERSCHEINUNGEN UND DRUCKVERHÄLTNISSE AN KLAPPENWEHREN, Untersuchungen aus dem Flussbaulaboratorium der Technischen Hochschule Karlsruhe. By H. Schwarzmann. Munich and Berlin, R. Oldenbourg, 1934. 41 pp., illus., diagrs., charts, tables, 11 × 8 in., paper, 3.80 rm.

This work records the results of extensive experiments with a model, which give a clear picture of pressure conditions in shutter weirs. On the basis of these experiments, the flow over weirs was investigated generally and the factors that determine the amount of flow were established.

DESIGN OF RESIDENTIAL AREAS, Basic Considerations, Principles, and Methods. By T. Adams. Cambridge (Mass.), Harvard University Press, 1934. 296 pp., illus., diagrs., charts, tables, 10 × 7 in., cloth, $3.50.

This new addition to the Harvard City Planning Studies amplifies the author's previous studies of residential problems. Here he discusses the most economical method of good living conditions and obtaining them. The book contains much of value on urban conditions and trends, the control of land subdivision, zoning, methods of planning, and similar items. A number of practical examples are described.

ELEMENTS OF WATER SUPPLY ENGINEERING. By E. L. Waterman. New York, John Wiley & Sons, 1934. 302 pp., illus., diagrs., charts, tables, 9 × 6 in., cloth, $3.50.

A textbook for beginners in civil engineering, covering the requirements to be met, sources of supply, methods of transportation and treatment of water, pumping, distribution systems, operation, maintenance, and methods of financing.

HIGHWAY DESIGN AND CONSTRUCTION. By A. G. Bruce. Scranton, Pa., International Textbook Co., 1934. 628 pp., illus., diagrs., charts, tables, 9 × 5 in., cloth, $4.

This is an admirable presentation of the fundamental principles of highway engineering in a single volume of moderate size. The text is concise, yet clear and comprehensive, and presents current general practice. Location, design, materials, surfacings, pavements, accessories, costs, street cleaning, and snow removal are discussed, and a final chapter is devoted to airport landing fields and runways. The author is senior highway engineer in the U. S. Bureau of Public Roads.

MEHRSTUFIGE RAHMENFORMELN. Bd. 1. Einführung, Anwendungsbeispiele, Belastungshilfsgrössen, Durchlaufbalken. By K. F. Sitte. Brünn, Rudolf M. Rohrer, 1934. 238 pp., diagrs., charts, tables, 8 × 6 in., paper, 8 rm.

This book is intended to facilitate the design of framed structures by providing a collection of rapid methods and simple formulas covering most of the conditions that occur in practice. The volume includes the general-method formulas for determining stresses in simple trusses and continuous beams, together with an introduction and numerous examples of their application.

NOTES ON THE HISTORY OF ANCIENT ROADS AND THEIR CONSTRUCTION. By R. J. Forbes. Amsterdam, N. V. Noord-Hollandsche Uitgevers-Mij, 1934. 182 pp., illus., diagrs., 11 × 7½ in., cloth.

This volume gives a survey of prehistoric and early roads in such countries as Malta, Crete, Egypt, Palestine, Western Asia, Persia, India, Greece, and the Roman Empire. It will be of interest to historians as well as to engineers because the construction of roads has in every country been inseparably associated with the development of civilization. Materials and methods of construction are described in considerable detail against a vivid historical background.

PIPE-LINE FLOW FACTOR IN THE HYDRAULIC FLOW FORMULA $\left(h = \frac{f}{d}\frac{v^2}{2g}\right)$ AND ITS RELATION TO DENSITY AND VISCOSITY. By F. M. Towl. New York, N.Y., Published by the author at 25 Broadway, 1934. 52 pp., diagrs., charts, tables, 9 × 6 in., cloth. $2.50 (Checks to be sent to author and made payable to Cornell University, where proceeds are to be used for hydraulic research).

This paper is intended to simplify the work of engineers interested in the transportation of fluids through pipe lines. The author discusses the coefficient of flow and its relation to steady flow through cylindrical pipes and to the Reynolds number. A slightly modified form of the flow formula is suggested, from which tables or curves that simplify the solution of pipe-line problems can be constructed. The practical applications of the modified formula are illustrated. A few free copies are available for hydraulicians.

TECHNIQUE OF MODERN WELDING. By Prof. P. Bardtke; translated from the 2d German edition by H. Kenney. London and Glasgow, Blackie & Son, Ltd., 1933. 299 pp., illus., diagrs., charts, tables, 9 × 6 in., cloth, 15 s.

This work aims to give a survey of welding technique that will be of use to engineers and also to provide a manual for the training of welders. Part I discusses welding processes; Part II, the applications of welding and the testing of welds; and Part III, gas cutting. The author is director of the welding repair shop and welding research department of the German State Railways.

TEXTBOOK ON HYDRAULICS. By G. E. Russell. 4 ed. New York, Henry Holt & Co., 1934. 447 pp., illus., diagrs., charts, tables, 9 × 6 in., cloth, $3.90.

A clear, logical presentation of the fundamental principles of hydraulics, intended for students and as a reference book for engineers. The results of recent research work are included, and references to sources are given. This edition has four new chapters on hydraulic turbines and pumps and an appendix giving a general treatment of the flow of fluids other than water in pipes.

WASSER- UND SINKSTOFF-BEWEGUNGEN IN FLUSS- UND SEEHÄFEN. Untersuchungen aus dem Flussbaulaboratorium der Technischen Hochschule zu Karlsruhe. By F. Rohr. Munich and Berlin, R. Oldenbourg, 1934. 52 pp., illus., diagrs., charts, tables, 11 × 8 in., paper, 4.80 rm.

This discussion of the flow of water and silt in harbors is based on experiments with a model harbor. Conditions governing such factors as the flow of water into and from the basin, eddy currents, and dead water were investigated and explained. An appendix contains extracts from two reports by Professor Rehbock, in which the method of model study is used to find methods of preventing silting in the harbor of Constitucion, Chile, and the Vulcan Harbor at Rotterdam.

WATER POWER ENGINEERING. By H. K. Barrows. 2 ed. New York and London, McGraw-Hill Book Co., 1934. 762 pp., illus., diagrs., charts, tables, 9 × 6 in., cloth, $6.

The principles and practice underlying the design of hydro-electric plants are described in a comprehensive way in this textbook, which is based on the course in the subject given at the Massachusetts Institute of Technology. Useful information on modern stations and their equipment is included, and many practical data are given. The new edition has been carefully revised, and the section on the cost and value of water power has been largely rewritten.